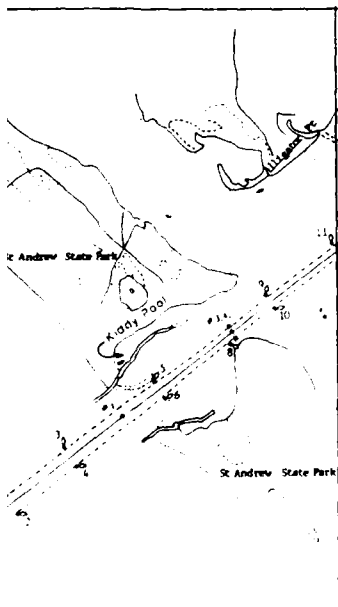




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TECHNICAL REPORT CERC-89-7

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A STUDY OF SAND WAVES IN THE PANAMA CITY, FLORIDA, ENTRANCE CHANNEL

by

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Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
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<p>The entrance channel to St. Andrew Bay, near Panama City, Florida, is subject to the formation of large sand waves. A study was undertaken to determine the extent of the problem, as related to navigation, and to assess possible mitigative alternatives. The inlet was originally constructed in the early 1930's and now must be dredged every 12 to 24 months as a result of the large sand waves which form in the navigation channel.</p> <p>The study considered structural changes to the inlet/jetty system using "INLET," a one-dimensional inlet flow model. Various alternatives were assessed to determine if structural modifications would help reduce sand wave sizes and/or decrease the necessary dredging frequency. Alternative dredging techniques were also investigated to identify more cost-effective methods of maintaining the sand wave prone navigation channel.</p>					
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PREFACE

The US Army Engineer District, Mobile (SAM), requested the US Army Engineer Waterways Experiment Station's (WES's) Coastal Engineering Research Center (CERC) to conduct a study of sand waves in the St. Andrew Bay entrance channel located near Panama City, Florida. Funding authorizations by SAM were granted in accordance with Intra-Army Order No. AD-86-3018.

The study was conducted at CERC under general direction of Dr. James R. Houston and Mr. Charles C. Calhoun, Jr., Chief and Assistant Chief, CERC, respectively; and under direct supervision of Mr. Thomas W. Richardson, Chief, Engineering Development Division (CD); and Ms. Joan Pope, Chief, Coastal Structures and Evaluation Branch (CD-S). The study was conducted by Mr. W. Jeff Lillycrop, Ms. Julie D. Rosati, and Mr. David D. McGehee, CD. Report figures were drafted by Messrs. Perry Reed and Leslie Wallace, CD. Report editing was performed by Ms. Shirley A. J. Hanshaw, Information Products Division, Information Technology Laboratory, WES.

Throughout the study, coordination was maintained with Mr. Pete Robinson or SAM. The Panama City Area Office assisted with field data collection and general site support through the following individuals: Messrs. Alton Colvin, Harry Peterson, Jack Branning, and "Gator" Brown.

Acting Commander and Director of WES during publication of this report was LTC Jack R. Stephens, EN. Technical Director was Dr. Robert W. Whalin.



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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
feet per second	0.5921	knots
inches	2.54	centimetres
knots (international)	0.5144444	metres per second
miles (US statute)	1.609347	kilometres
square miles	2.589998	square kilometres
tons (2,000 pounds, mass)	907.1847	kilograms

A STUDY OF SAND WAVES IN THE PANAMA CITY, FLORIDA, ENTRANCE CHANNEL

PART I: INTRODUCTION

Background

1. At the request of the US Army Engineer District, Mobile (SAM), a study of sand waves in the Panama City Entrance Channel was conducted by the Waterways Experiment Station's (WES's) Coastal Engineering Research Center (CERC). Sand waves in the entrance channel to St. Andrew Bay are a navigation hazard and a maintenance problem. Although ships rarely bump bottom, sand waves can grow large enough to reduce authorized channel depths. Within about 18 months after dredging, sand waves having heights as great as 15 ft* are present and must be removed. To maintain the inlet channel, frequent over-depth dredging is required.

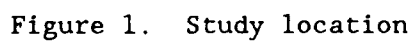
2. Little is known about the formation and migration of sand waves. What is known comes mostly from laboratory flume test results of sand ripples to explain the much larger bed forms. From flume tests it is known that sand waves form only under certain flow conditions and require a sufficient sand source. To mitigate sand wave formation and increase the maintenance dredging interval, flow velocities must be modified and/or the sediment supply reduced.

3. In an effort to reduce dredging requirements, tests were conducted to study potential changes to the inlet to modify existing flow conditions and to determine methods for reducing the amount of sediment entering the channel. A complete background investigation of the study area was performed, including a review of all available literature. An evaluation of inlet hydraulics was conducted and a computer model used to simulate inlet hydraulics. Sediment sources were identified and various alternatives were considered to reduce the amount of sediment entering the channel from these sources.

Study Location and General Conditions

4. The study site is located at the entrance channel to St. Andrew Bay, Panama City, Florida (Figure 1). Comprising an area approximately 100 miles

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.



from Pensacola, Florida, and about 100 miles from the Florida state capital at Tallahassee, the St. Andrew Bay system consists of four adjacent bays: St. Andrew Bay, West Bay, East Bay, and North Bay (Table 1). For purposes of this report, St. Andrew Bay will be used to refer to the complete system, including all four bays, unless otherwise specified. The area of the bay system is approximately 108 square miles.

Table 1
Bay System Size

<u>Bay</u>	<u>Surface Area</u> <u>square miles</u>	<u>Mean Depth</u> <u>mlw*</u>
West	27.5	7.7
North	10.4	8.3
St. Andrew	40.9	15.5
East	29.2	12.4

* Mlw = mean low water.

5. The entrance channel and jetties, completed in 1934, separate Lands End Peninsula from what is now Shell Island. The channel is a federally maintained navigation channel consisting of a 450-ft wide, 32-ft-deep (mlw) approach channel from the gulf. The channel narrows to 300 ft wide about half-way through the inlet throat but remains 32 ft deep into the bay. The inlet is stabilized by two stone jetties spaced 1,500 ft apart.

6. The complete Panama City Harbor Navigation Project consists of the entrance channel, a small channel in Grand Lagoon, and the bay channel which crosses St. Andrew Bay and connects to the Panama City port facilities. The complete project, including authorized channel dimensions, is shown in Figure 1. Land adjacent to the inlet is owned by the State of Florida, and the west side of the inlet has been developed for recreational use.

7. The inlet cross section is generally shallower on the east side, with depths averaging about 6 to 12 ft. Actual navigation channel depth varies with location and condition of the sand waves but generally is about 30 to 35 ft. Along the west side of the inlet the natural inlet thalweg has created a scour problem along the jetty, generating depths as great as 45 to 50 ft.

8. Sediment within the study area consists of fine- to medium-grained

Table 2
Sediment Analysis of Entrance Channel

<u>Sample Number</u>	<u>Coarse Sand</u> %	<u>Medium Sand</u> %	<u>Fine Sand</u> %	<u>Silt Sand</u> %
1	4	74	15	7
2	3	52	38	7
3	-	15	76	9
4	1	39	52	8
5	1	49	40	10

sand. Table 2 presents an analysis of sediment samples taken from the navigation channel center line (Figure 2) within the inlet throat for a draft environmental statement (US Army Corps of Engineers (USACE) 1975). Bottom

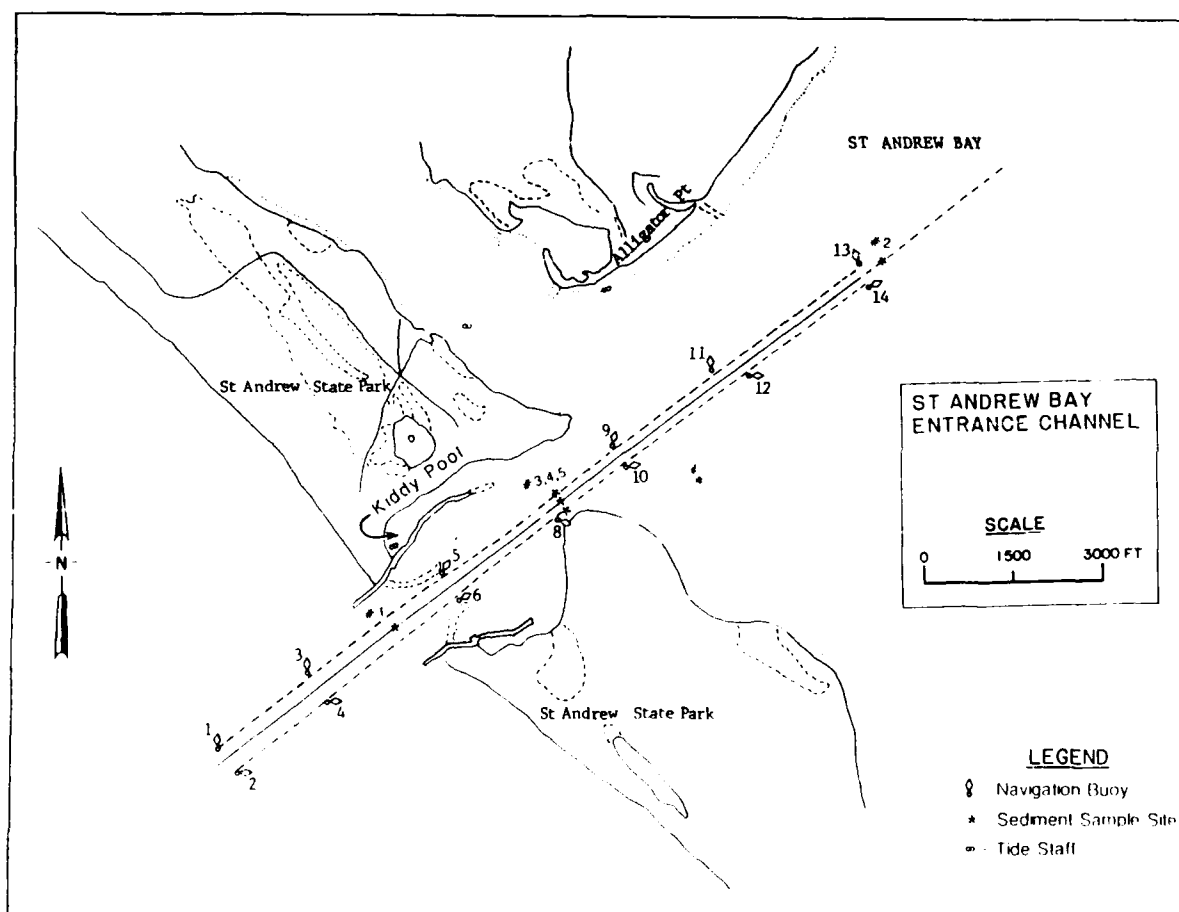


Figure 2. Location map

sediment in the Panama City Entrance Channel is characterized by quartz sand with grain size varying between 0.2 mm to approximately 0.35 mm.

9 Erosion problems resulted in construction of rubble-mound jetty wings which extend back from the jetties toward the bay to help stabilize the shoreline. Erosion from behind the jetty wings has created protected areas on either side of the inlet. The area on the west side of the inlet is used by the park as a swimming area and is referred to as the "Kiddy Pool."

10. Average wind speeds and directions for each month are presented in Table 3 (see Appendix C of Beach Erosion Control and Hurricane Protection, Interim Feasibility Report for Panama City Beach (SAM 1976.)) The data used to prepare the table were collected between 1884 and 1968, the majority being between 1954 and 1968.

11. Offshore wave heights for the area average 2 to 4 ft based on long-term averages by the National Climatic Data Center using Shipboard Synoptic Meteorological Observations (SSMO) (SAM 1976). However, during storms heights of up to 15 ft have been recorded. Hurricanes impact the area with a frequency of occurrence being one storm every 3.8 years.

Table 3
Average Wind Speed and Direction

<u>Month</u>	<u>Wind Speed*</u>	<u>Prevailing Direction</u>
January	15.3	East
February	15.8	Northeast
March	13.9	Southeast
April	12.8	Southeast
May	10.1	East
June	9.6	East
July	9.2	Southeast
August	9.7	East
September	13.9	East
October	14.6	Northeast
November	14.8	East
December	15.4	East
Mean Annual	12.9	East

* In miles per hour.

Statement of the Problem

12. Since the inlet was completed in 1934, shoaling has been a continuous problem. The Panama City Harbor Federal Navigation Project has been dredged 26 times in the past 54 years, totaling over 10.6 million cu yd (approximately 200,000 cu yd annually). In addition to the shoaling problems, jetties built to stabilize the inlet have undergone continuous degradation. The west jetty has required rehabilitation many times because of deep scour holes along the inner jetty and near the seaward end.

13. Primary shoal formation consists of sand waves which are typically perpendicular to the channel through the entire inlet throat and into the bay. Heights of these sand waves vary from as large as 15 ft near the tips of the jetties to only a few feet within the bay. A more complete description of these bed forms is presented in Part III. Because of the size and relatively quick reformation rate of the sand waves, the inlet must be dredged approximately every 18 months.

14. The federally authorized navigation project provides for a channel depth of -32 ft mlw, but to cope with the sand waves problem, the inlet is typically dredged to -40 ft mlw. The inlet was last dredged in June 1986; however, a fathometer survey of the inlet 4 months later (the following October) showed sand waves had reformed with heights ranging between 6 to 8 ft. Another fathometer survey in July 1987 showed sand wave heights ranged between 8 to 12 ft, the largest being near the tips of the jetties. The inlet is again scheduled for maintenance dredging near the beginning of the 1988 calendar year.

15. Effective maintenance of the navigation channel requires reducing the size and/or the formation rates of the sand waves. Reducing the amount of sediment reaching the sand waves will slow their formation rates after dredging, thus increasing maintenance dredging intervals. Reducing sediment transport into the inlet requires identification of the primary sediment sources followed by modifications to reduce the amount of sediment entering the entrance channel. Part IV addresses the sedimentation problem and discusses potential solutions to reduce sediment transport into the navigation channel.

16. Because bed form size is strongly related to current velocity, reducing the height of the sand waves will require modifications which change bottom current velocity. Figure 3 shows a generalized relationship between

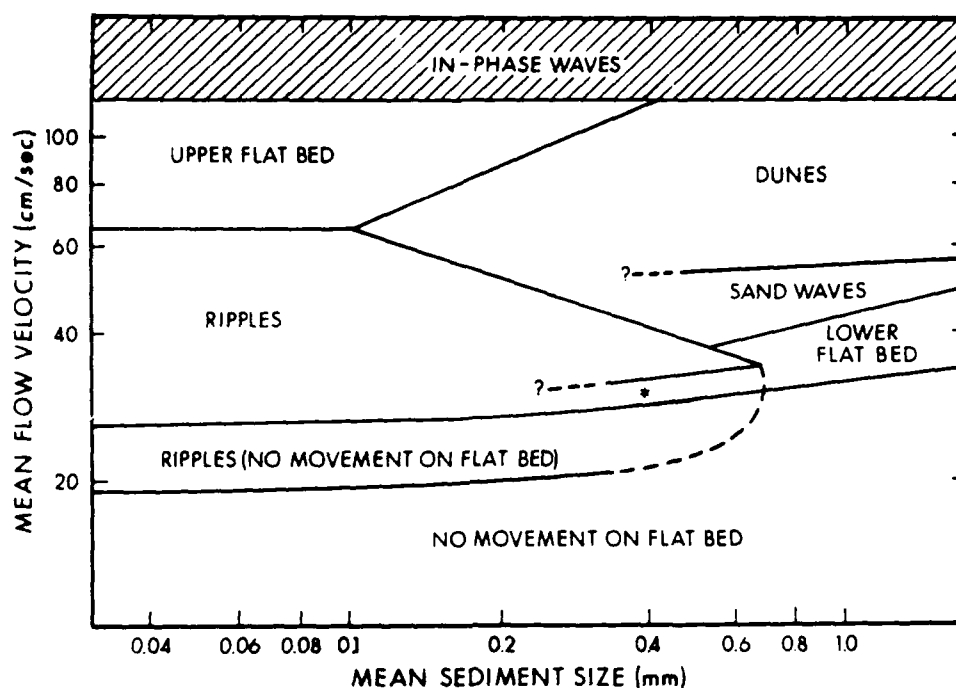


Figure 3. Flow versus sediment relationship (SEPM 1975)

mean sediment size, mean flow velocity, and characteristic bed forms based on two-dimensional laboratory studies (Society of Economic Paleontologists and Mineralogists (SEPM) 1975). Threshold velocity for sediment movement is approximately 20 cm/sec, and formation of each bed form depends on sediment size and velocity. Boundaries in this figure are approximate based on laboratory flume experiments and are not a completely accurate representation of the larger scale processes found in nature.

17. The jetties and jetty wings have required continuous maintenance since they were completed in 1934. Since 1950 the jetties have required over 65,000 tons of stone for maintenance (Figure 4). Because of their potential role in mitigating sand wave problems in the inlet, they have been considered in this study.

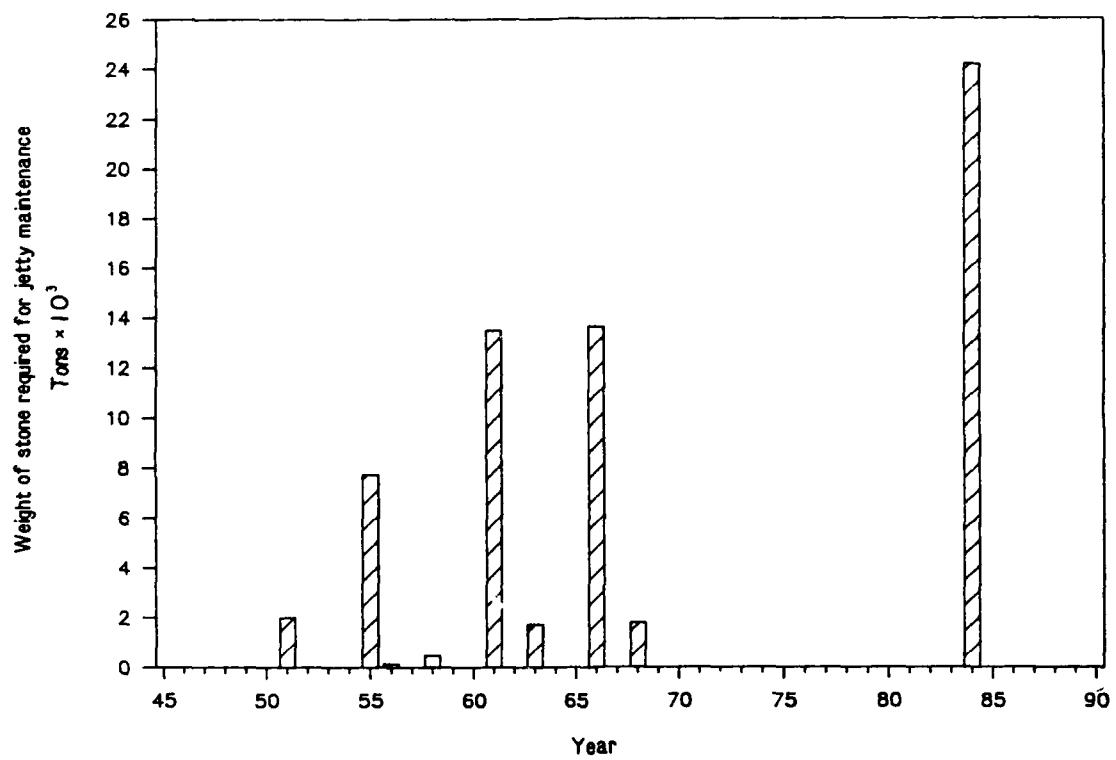


Figure 4. Jetty maintenance at Panama City, FL

PART II: SAND WAVE CHARACTERISTICS

18. Little is known about the formation and migration of sand waves in a natural setting. Basic prototype theory comes from limited field observations and extension of laboratory flume tests of sand ripples to explain the much larger bed form. From flume tests it is known that sand waves will not form without sufficient sand supply and that certain flow conditions must exist before ripples begin to form and migrate. Also, once threshold of movement (critical) velocities are achieved, further increases in velocity produce higher migration rates. Because laboratory studies are generally conducted at much smaller scales than exist in nature, extrapolation of theory from lab to nature is inexact. A very useful product of these studies lies in the identification of the primary factors involved in the formation of sand waves. As stated previously, for sand waves to form there must be (a) a nearly continuous supply of sand and (b) currents of sufficient strength and duration to cause sediment motion. Without both of these elements sand waves will not form. Since sand waves do form in the Panama City Entrance Channel, both of these must be present.

19. As flow velocity over a flat erodible bed increases, sediment begins to move and small ripples form. These ripples are generally long crested and two-dimensional, and they face and move in a downstream direction. Figure 5 shows a generalized bed configuration for these conditions. As flow velocities further increase, crests divide and the bed becomes three-dimensional. Ripples are small, having heights of only a few centimeters and crest-to-crest lengths on the order of 10 to 20 cm.

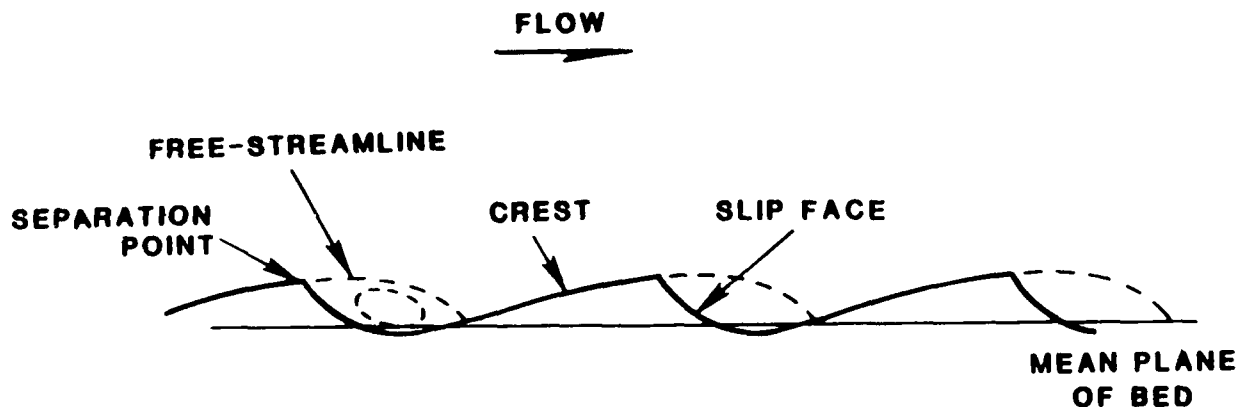


Figure 5. Bed-form characteristics

20. With further increases in flow velocity, larger bed forms appear. These are often called megaripples and are larger than ripples but smaller than sand waves such as those in the entrance channel at Panama City. Megaripples are also long crested and two-dimensional, and they face and move in the downstream direction. Megaripples have heights ranging from 30 cm to several meters and wave lengths up to a few tens of meters. Ripples are often found on the upstream faces of the larger megaripples.

21. Sand waves form as flow velocities increase further. They are larger than the other bed forms previously described and are not as closely spaced. Their heights have been reported to be as great as 12 m with wavelengths of hundreds of meters. However, sand waves found in the Panama City channel range between 2 to 5 m in height with wavelengths between 30 and 60 m. The sand waves considered in this study are subjected to an oscillatory tidal flow with change of direction occurring about every 12 hr. However, the path of the sand waves tends to be unidirectional with shape and advance rate determined by the strongest current speed and/or duration. The influence of current duration is not as well quantified as the other parameters considered in describing formation of sand waves.

PART III: LITERATURE REVIEW

22. A literature search was conducted to review studies concerning coastal processes in the vicinity of St. Andrew's Bay. Of particular interest was literature discussing aspects of sand waves at the site. This section briefly summarizes background information available and focuses on a study by Salsman, Tolbert, and Villars (1965) titled "Sand Ridge Migration in St. Andrew Bay, Florida."

23. Stapor (1973) discusses the recent (since 1779) evolution of Shell and Crooked Islands (Figure 6); and by comparing bathymetries of 1877, 1930 and 1946, he estimates the rate of longshore transport in the region. The rate is estimated to be almost evenly balanced, with approximately 153,000 cu yd/yr to the west and 141,000 cu yd/yr to the east, for a net transport rate of 12,000 cu yd/yr to the west during the period of the study.

24. A study by Wang et al. (1978) estimated rates of longshore sediment transport at Panama City Beach located just west of St. Andrew Bay (Figure 1). Transport rates were estimated using two methods: by measuring the quantity of material trapped at a littoral barrier (groin) intermittently from August 1975 to October 1977 and by a sand tracer experiment conducted from 18 to 24 October 1977. Because the groin was neither long nor high enough to intercept all longshore-moving material, the authors believe that the estimated net quantity (6,000 cu yd/yr) was too low. The sand tracer experiment suggested rates of 510,000 cu yd/yr to the west and 300,000 cu yd/yr to the east, for a net sediment transport rate of 210,000 cu yd/yr to the west for the mid-October 1977 time period.

25. Other studies which have included estimates of the rate of longshore sediment transport in the vicinity of the St. Andrew Bay entrance channel include Walton (1979) and SAM (1976). Walton estimated sediment transport potential, based on SSMO data, to be 219,000 cu yd/yr to the west and 192,000 cu yd/yr to the east, for a net westerly transport of 27,000 cu yd/yr. SAM, using the littoral drift rose method (Walton 1973), estimated a total gross sediment transport potential of 413,000 cu yd/yr for a net westerly transport of 6,800 cu yd/yr.

26. Ichiye and Jones (1961) measured temperature, salinity, and currents over several 24-hr periods for five seasons between 1957 and 1959 in the St. Andrew Bay system. The velocity data in the channel were measured as

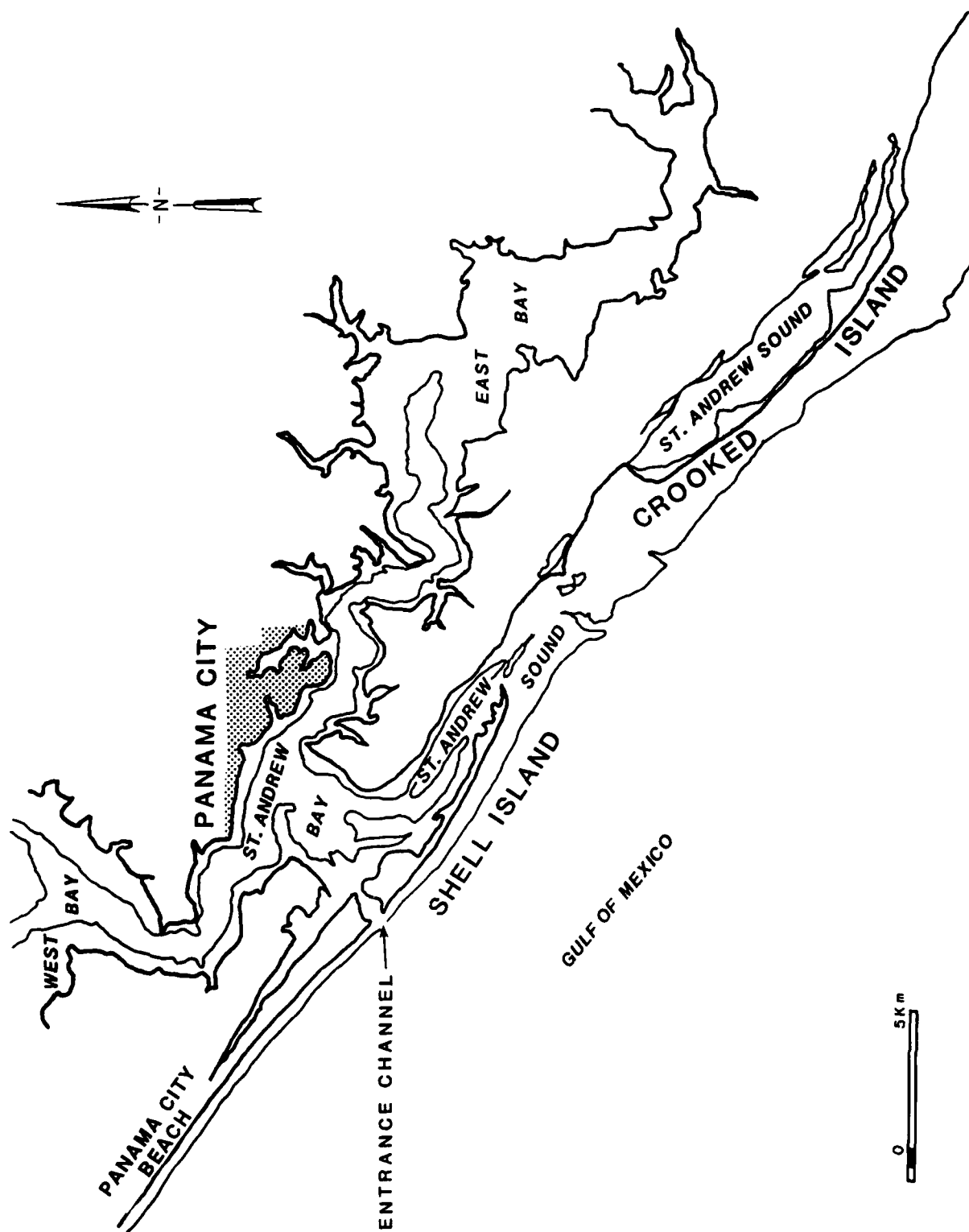


Figure 6. St. Andrew Bay and vicinity

1.7 and 2.0 ft/sec, for the flood and ebb currents, respectively. Current velocities as low as 0.023 ft/sec (0.7 cm/sec) could move fine sand and silts.

27. Several studies discuss erosion of Panama City Beach located west of the St. Andrew Bay entrance channel (e.g., Saloman et al. 1982, Culter and Mahadevan 1982). According to results of these studies, the erosion of this area is most likely due to partial blockage of westward-moving littoral material at the St. Andrew Bay entrance channel.

28. Salsman, Tolbert, and Villars (1966) document an investigation of the sand ridges in St. Andrew Bay. The fine grained (0.135-mm) sandy bottom undulations were discovered in the fall of 1982 and were located in 11 m of water in an area marked "mud" on nautical charts. Prior to the St. Andrew Bay cut in 1934, Shell Island protected the bay from high currents, and only finer sediments could be transported and deposited in the bay. However, the new entrance resulted in higher currents and wave energy in the bay, and it could now transport sand-sized material. Two distinct sediment regimes were found in the bay during an earlier study (Waller 1961): sandy areas near the entrances and in all areas shallower than 6 m and silty clay areas in deeper water. The sand ridges were monitored for over 2 years. The ridges were perpendicular to the channel axis, being from 30 to 60 cm in height, with wave lengths of 13 to 20 m. The ridges were asymmetric in shape, with steep slopes facing down current. The ridges moved during the predominant flood current, and the rate of migration was estimated to be 1.35 cm/day. The authors estimate that each ridge left behind a layer of sand averaging 12 cm in thickness as it passed a particular point.

29. Reviewed studies indicate that the rate of net longshore sediment transport in the vicinity of St. Andrew Bay entrance channel is relatively low, with a slight east-to-west dominance. The man-made cut in Shell Island, creating the St. Andrew Bay entrance channel, allows higher currents and waves to transport sand into the channel and the bay. Sand is deposited in the channel and bay during flood currents, removing littoral material from the system and thereby creating erosion problems for adjacent beaches.

PART IV: PROJECT SITE EVALUATION

Inlet History

30. Literature from the Panama City Area Office and SAM provide extensive information about the construction and maintenance history of the inlet and jetties. This section summarizes the extensive work which has been performed during the past 54 years. Prior to construction of the existing navigation channel through Lands End Peninsula, the harbor entrance was through East Pass (Figure 1), approximately 7 miles east of the existing channel. This channel was described as "torturous" in a letter report* from the SAM District Engineer to the Gulf of Mexico Division Engineer in New Orleans and, in fact, was not used at night or during storms by deeper draft vessels. East Pass was maintained with a channel depth of 22 ft and width of 200 ft. From the gulf entrance of East Pass to the Panama City harbor facilities was a distance of about 5 miles. It was because of this long and dangerous channel that "New Pass" was constructed and completed in 1934.

31. The new channel project was 10,500 ft long with approximately 3,000 ft through Lands End Peninsula and dry land. The cut divided the Lands End Peninsula and created Shell Island. Two riprap jetties were built to stabilize the new cut. They extended gulfward to the 12-ft contour and were built on 2-ft-thick rock mattresses with steel sheet-pile cores along the jetty axis. The east jetty, from the low water line extended 500 ft offshore and the west jetty, from the low water line, extended 550 ft offshore. Both jetties had crest elevations of +6 ft mlw and crest widths of 8 ft and were spaced 1,500 ft apart. Original construction of New Pass included rubble-mound jetty wings with steel sheet-pile cores extending landward 300 ft.

32. Soon after project completion the channel banks, especially at the landward end of the jetties, began to experience substantial erosion caused by gulf waves. In 1935 steel sheet-pile bulkheads consisting of a single row of 15-ft piles were built into the channel and reinforced with steel sheet-pile groins and buttresses, all having a crest elevation of +2.5 ft mlw. The bulkheads were 800 ft and 1,050 ft long for the east and west jetties,

* Letter report titled "St. Andrew Bay Entrance Channel," April 1929, from COL R. Park, District Engineer, to Division Engineer, Gulf of Mexico Division, New Orleans, LA.

respectively. Work on the bulkheads was completed in May 1935. Within 6 months the west bulkhead was destroyed, and the east bulkhead was significantly damaged by unspecified causes. Repairs to the east section were made by placing large stone in front of the structure. Not long after this too failed, and in 1936 the large stones were removed.

33. In July 1936 a hurricane caused catastrophic damage to the jetties. Wing crest elevations went from +6 ft mlw to approximately +1 ft mlw. Extensive repairs were made in 1937 and early 1938. Reinforced asphaltic concrete mats, 2 in. thick and extending out 24 ft from the toe of the existing rock jetties and wings, were placed and anchored with large precast asphaltic three-sided forms, or prisms. Hot asphaltic concrete was formed and smoothed over the existing jetties. The crest elevations were restored to +6 ft mlw, and the crest width was 8 ft. A complete pictorial record of this restoration at the Panama City Area Office provides a fascinating look at the jetty reconstruction project nearly 50 years ago.

34. Flanking by waves of the west jetty was causing erosion problems; consequently, in 1939 the jetty wings were extended. The next year the wings were repaired, and 2 years later in 1942 the wings were lengthened by 680 and 800 ft for the east and west jetties, respectively. An inspection of the jetties in 1945 revealed that they were in relatively good shape except for some settling at the head of the west jetty.

35. Between 1934 and 1946 the channel banks suffered significant erosion and receded over 900 and 1,200 ft on the east and west banks, respectively. As a result of the shoaling that occurred, the channel was dredged nine times between 1935 and 1945. Table 4 summarizes information obtained from the area office and gives the year and volume of material removed during this 11-year period. As can be seen from the table, shoreline change during this period was dramatic. Figure 7 shows a shoreline change map with a precut shoreline (1855), including a small lake that was drained during initial construction, the postconstruction shoreline immediately following project completion (1934), and the 1945 shoreline. The location of the jetty wings in relation to the 1934 shoreline is also depicted.

36. In May 1946 a report* was prepared by the Shore Protection Board

* Letter report titled "Report on the Effect of Proposed Channel Improvements at St. Andrew Bay, Florida," May 1946, War Department, Shore Protection Board, Washington, DC.

Table 4
Dredging Events Between 1934 and 1945

<u>Year</u>	<u>Volume of Material cu yd</u>
1935	1,295,894
1936	0
1937	956,342
1938	457,501
1939	338,159
1940	262,522
1941	386,229
1942	491,762
1943	0
1944	0
1945	279,171
Total	4,467,580

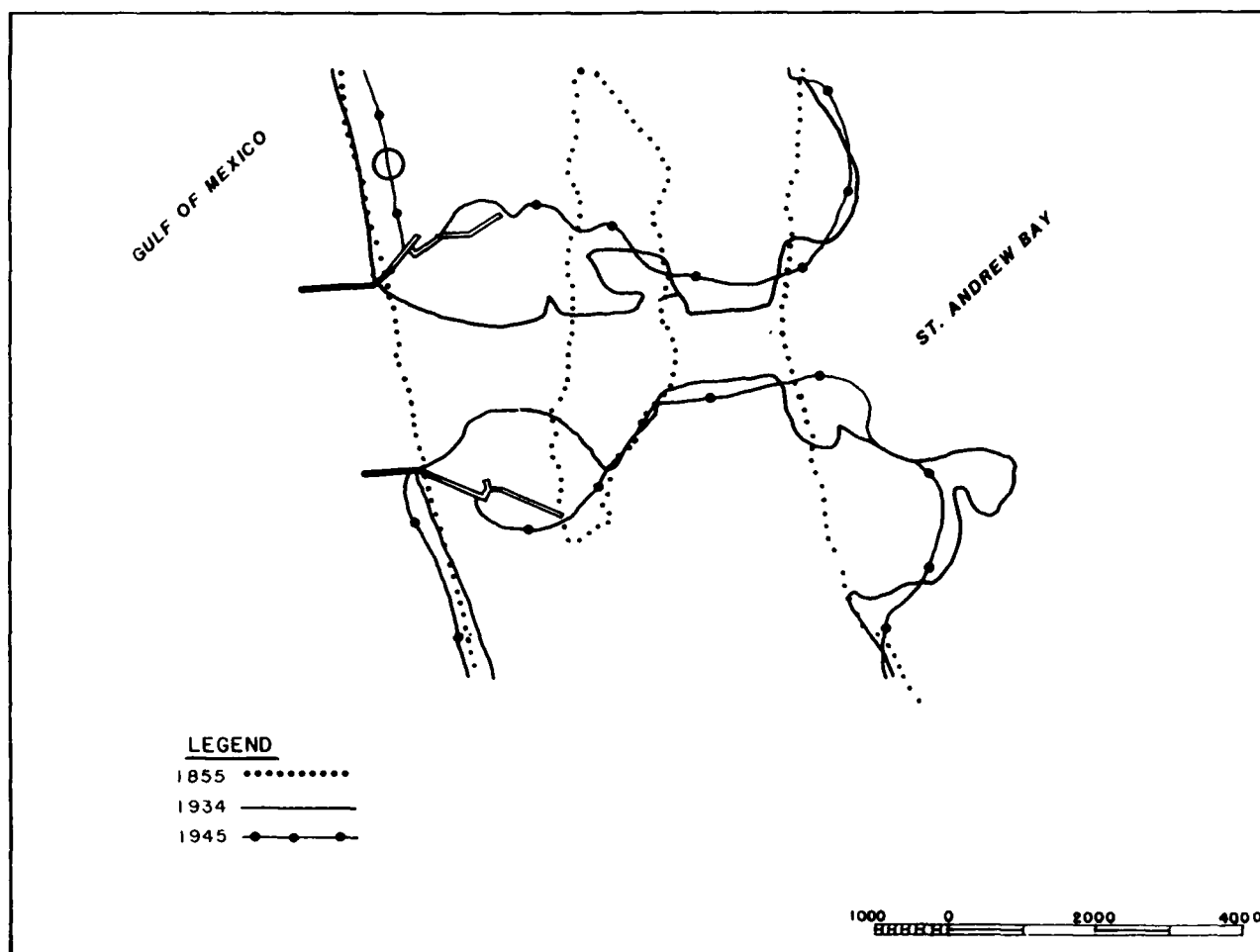


Figure 7. New channel entrance shoreline changes (1855 to 1945)

of the War Department which addressed the channel bank erosion problem. The report pointed out that even after the jetty wings had been lengthened, erosion and channel shoaling was still a serious problem. The cause of the erosion was reported to be incident waves from the gulf. Two possible solutions to this problem were presented: extend the jetties seaward to reduce incident wave heights reaching the channel banks or revet the channel banks through New Pass.

37. The problem with these short jetties was first discussed in a 1938 report* which stated that "the insufficiency of the length of the jetties and the divergence of the wings permit gulf storm waves to enter and cross the waterway and impinge upon the jetties and unprotected shoreline causing progressive widening of the cut." The report recommended the jetties be extended at least 1,500 ft.

38. A later report** considered the erosion along the west bank of the inlet and, more specifically, improvements to the west jetty wing to reduce this erosion. The Florida Park Service sponsored the study because they were interested in maintaining a recreational beach along the west shore of the inlet. The study included limited physical model tests to evaluate various jetty heights, permeability, and vertical structures behind the jetty. The report concluded that "it will involve not insignificant construction work to improve the existing jetty" and recommended periodic nourishment of the shoreline for maintaining a recreational beach.

39. Since construction of the inlet in 1934 the harbor project has been dredged 26 times, removing over 10.6 million cu yd of material (Figure 8). Unfortunately it is not possible to determine how much of that volume is directly from sand wave excavation because records only stated that dredging occurred for the Panama City Harbor navigation project and did not relate volumes to location.

Field Data Collection

40. The field data collection effort had the following objectives:

* See note on p. 17.

** Unpublished report titled "Model Study for the Improvement of the Jetties of the St. Andrew Bay Entrance Channel," December 1958, Coastal Engineering Laboratory, University of Florida, Gainesville, FL.

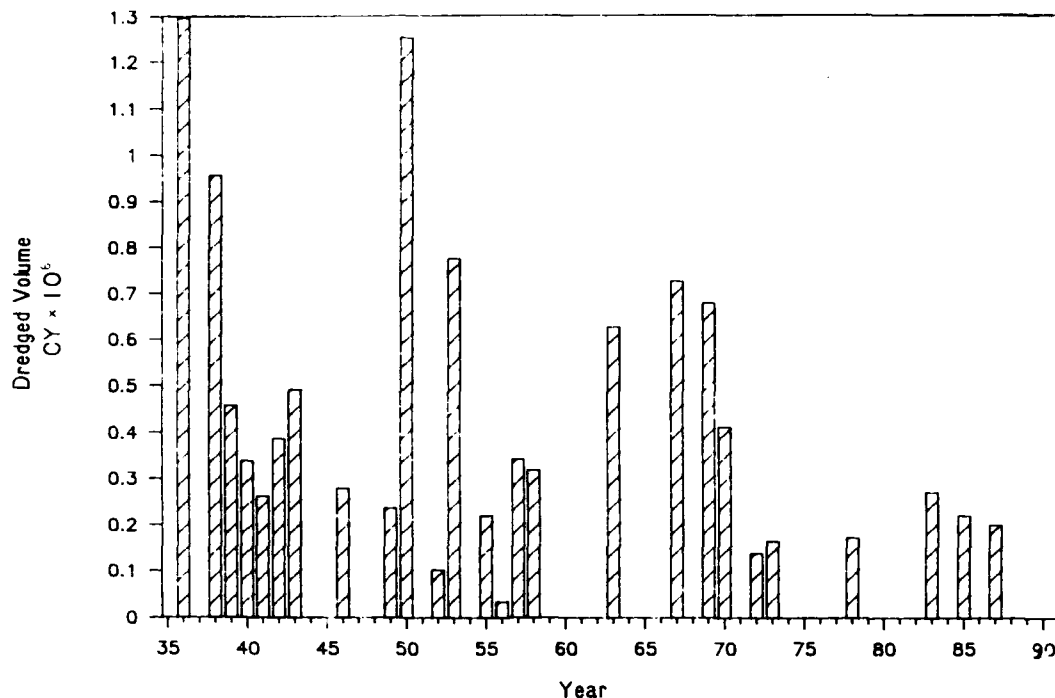


Figure 8. Panama City Harbor dredging events

- a. Qualitatively monitor sand wave formation through time.
- b. Determine hydraulic characteristics of the inlet necessary for the computer model.
- c. Measure the velocity distribution associated with a fully developed bed form at the prototype site.

41. These objectives were met with the following monitoring elements:

- a. Fathometer and side-scan sonar surveys.
- b. Deployment of in situ and profiling current meters over several tidal cycles.

Bathymetric surveys

42. Bathymetric surveys using acoustic sounders (Raytheon fathometers) provided sufficient information from the routine channel condition surveys. These were used to identify individual features and their approximate distribution within the inlet. Five parallel survey lines spaced approximately 100 ft apart were used to monitor the sand waves. These were the east channel edge, the center line, the west channel edge, and two lines between the centerline and the two channel edges. Surveys were made on 16 October and 6 November 1986 and on 10 July 1987.

43. All surveys were conducted using the Panama City Area Office

survey vessel "McBride" using a fathometer recording at a chart speed of 4 in./min. Positioning of the vessel during data collection was approximate, based on visual locations of the existing navigation aids and landforms as event marks on the record. Tidal datum correction was obtained by reading a staff gage located at the State Park at the beginning and end of each survey.

44. Height and spacing of the wave forms are quite evident in the bathymetric record (Figure 9). Development of individual features is indicated during successive surveys. Less evident in a series of discrete bottom profiles is the overall extent and horizontal orientation of the sand wave field. Since a composite plan view of the inlet would require numerous survey lines with accurate position control, an alternative is side-scan sonar to obtain a continuous picture.

Side-scan sonar

45. Side-scan sonar uses phased transducer arrays mounted on a towfish to emit acoustic pulses in narrow beams to each side. Timing of the return echoes permits computation of the slant range, perpendicular to the direction of towfish travel, to targets in the plane of the beam. Repeatedly pulsing the signal as the towfish is pulled forward generates a picture of the sea-floor as a series of scan lines on a moving chart recorder. Stronger returns show darker images, and a lack of any signal appears white. The result is an acoustic image of the bottom as seen from the position of the towfish (Clausner and Pope 1988).

46. When interpreting side-scan sonar imagery that is not dimensionally corrected by post signal processing, some allowance must be made for inevitable distortion. A true plan projection is not obtained since dimensions perpendicular to the tow path are slant ranges from the fish, while parallel dimensions are affected by the interrelation of chart speed to vessel speed. Nevertheless, it is a valuable qualitative tool for subaqueous investigations. In this study, side-scan surveys were conducted on 6 Nov 1986 from the "McBride" using a Klein side-scan sonar operating at 100 and 500 khz. Tow speed was between 3 and 4 knots with the towfish between 6 and 18 ft below the surface.

47. Figure 10 is a 100-khz image of the western edge of the inlet, extending from outside the jetties to the north end of the inlet. Notable features include the toe and slope of the west jetty tip (A) (the rock providing a strong return signal); a steep sand wave with a trough at the south

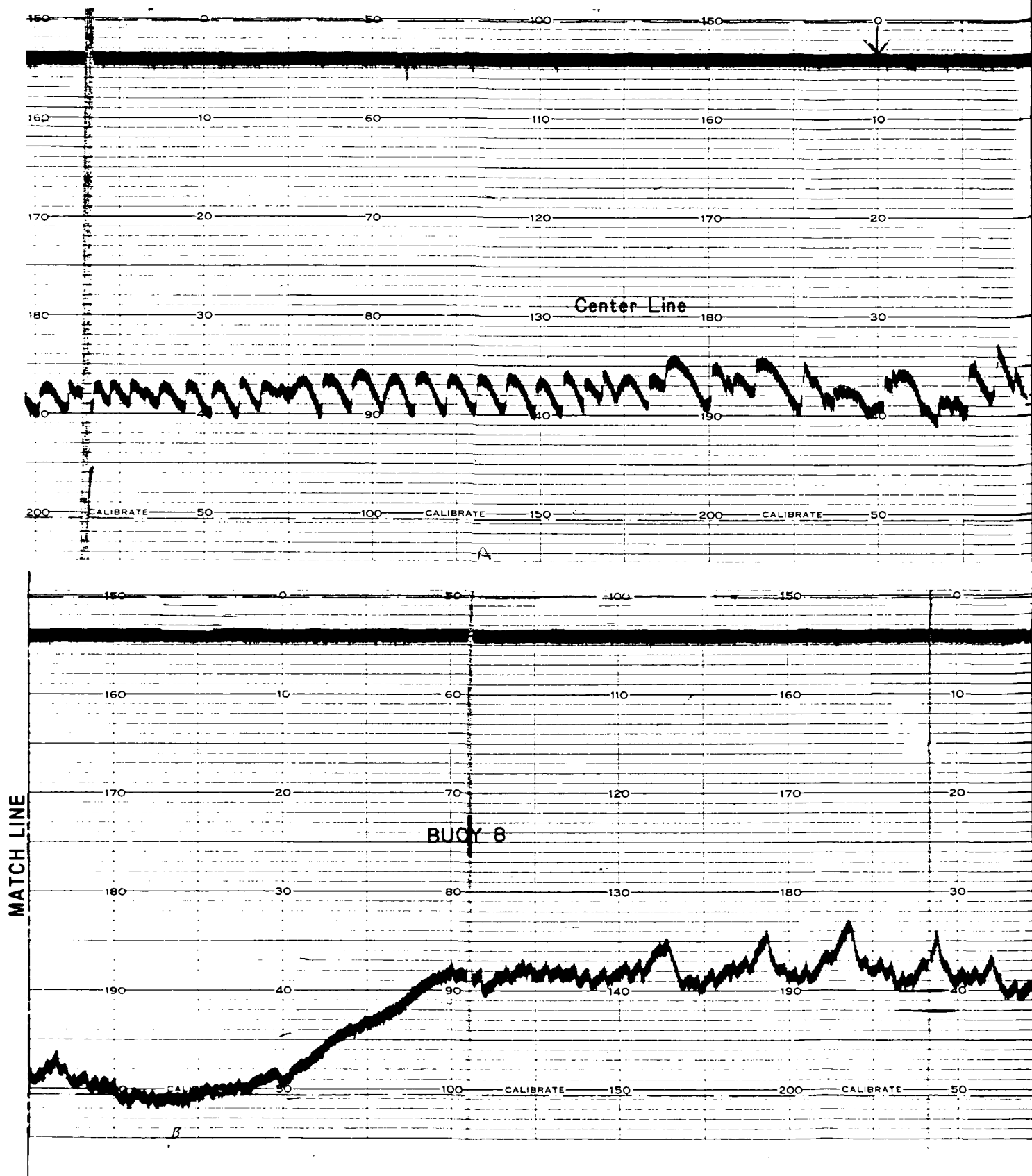


Figure 9. Bathym

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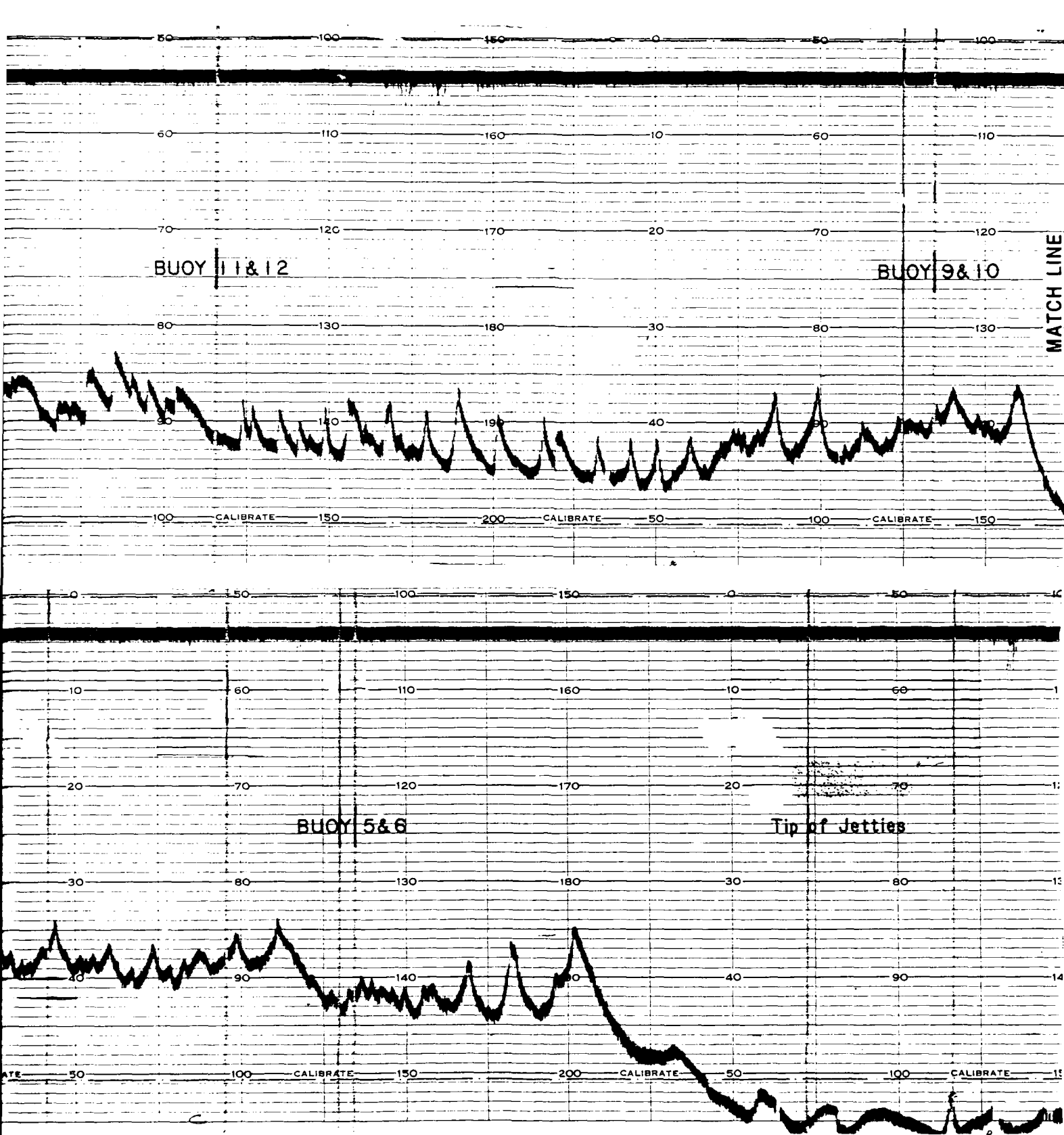
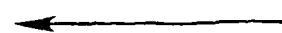


Figure 9. Bathymetric record

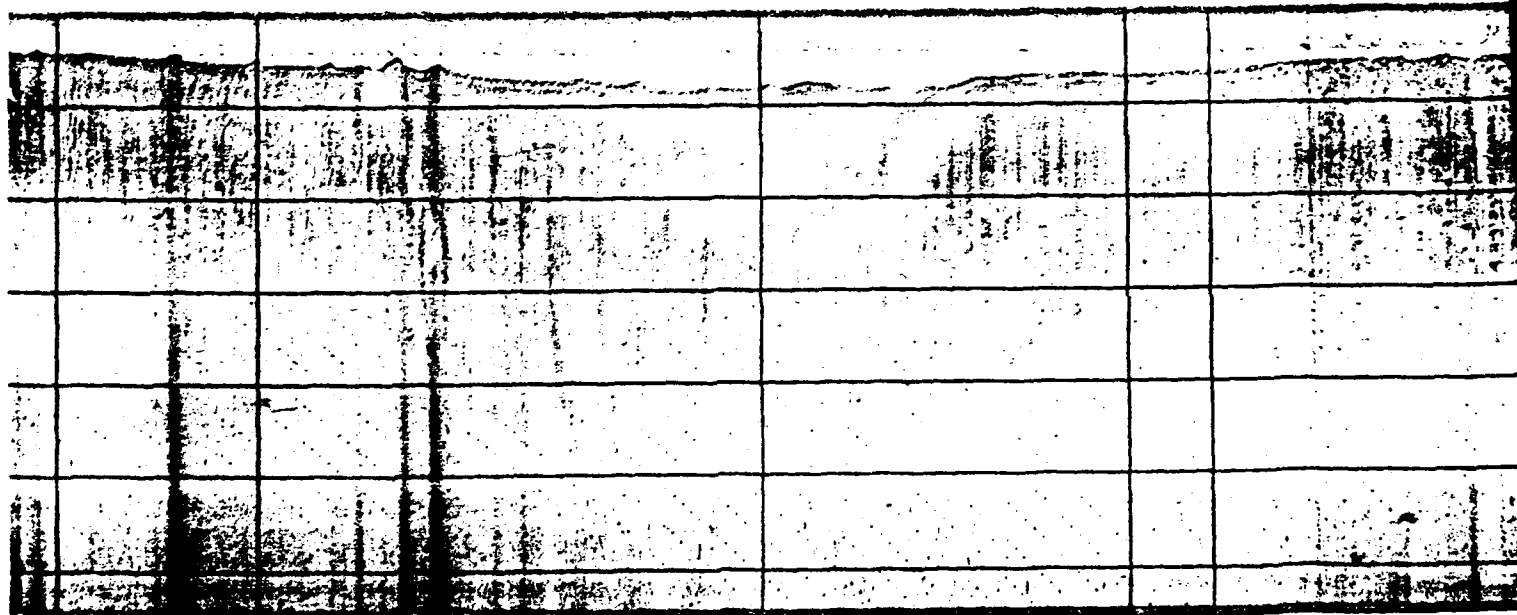


BAY

BUOY 9 & 10

BUOY 8

D



Y

GULF

Y 8

BUOY 5 & 6

D

C

D

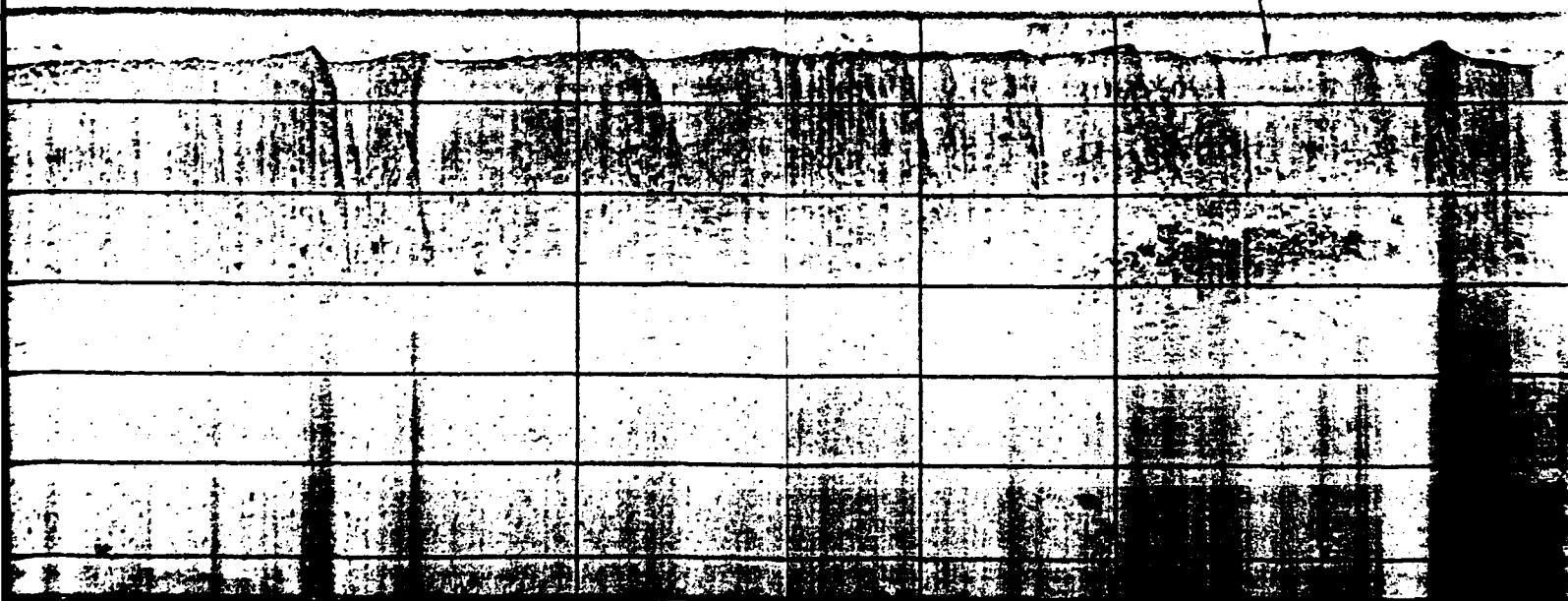


Figure 10. Side-scan sonar image

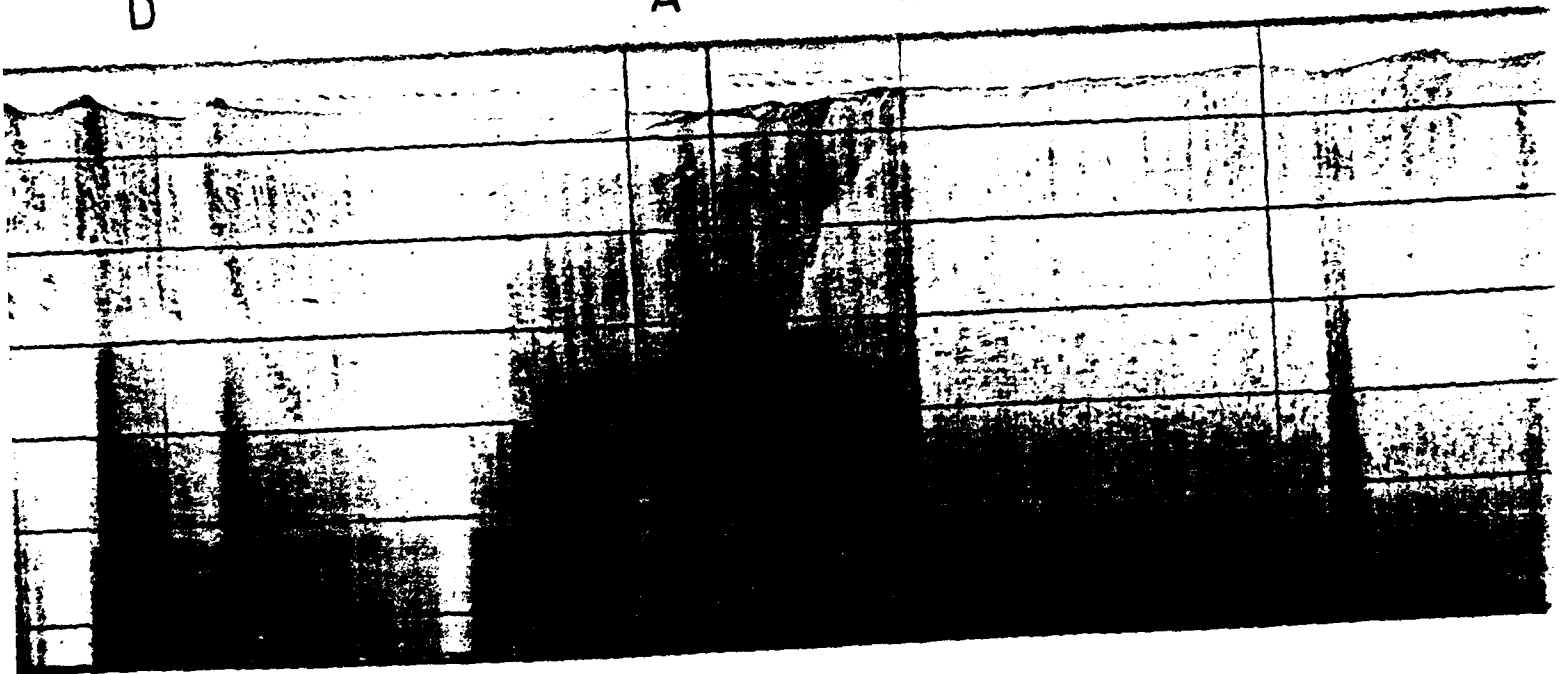
100 kHz Side Scan Survey

TIP OF JETTIES

D

A

B



side of the jetty tip (B); the western extension of a submerged retaining wall near buoy #5 (C); long-crested sand waves in several distinct groups through the inlet (D); and less regular waves, including several parallel to the inlet, in the vicinity of buoy #11 (E).

Current meters

48. Inlets are frequently modeled with open channel flow equations that assume fixed "walls" characterized by a friction factor resulting from micro scale roughness. The presence of sand waves indicates an interaction between water velocity and the bottom creating nonsteady state flows and boundary conditions. However, measurements taken at one or two near bottom stations can be indicative of the prevailing macroscale conditions without the expense of a complete spatially distributed time series data set. By placing in situ meters at sites with and without sand waves, the precipitating threshold velocities for formation can be estimated.

49. Before in situ current meters for data collection over several tidal cycles were placed, some preliminary data were obtained on 18-19 April 1987 to ascertain the maximum tidal induced flow and the variation of near-bottom velocities near sand wave crests and troughs. Measurements were made with an Endeco model 110 profiling current meter which uses a calibrated, ducted impeller for velocity and magnetic compass for direction. The in situ meters, deployed from 10 to 13 July 1987, were Endeco model 105 ducted impeller meters with internal recording capability. For ease of installation and protection from significant fishing boat traffic, gages were mounted on subsurface with taut moor buoys anchored to existing navigational buoy sinkers. The values recorded were at either 15- or 30 min intervals depending upon the specific gage. Gulf and bay water level differences were measured by manual recording of tide levels on staffs placed at three locations (indicated in Figure 2) during the daylight hours of 11 and 12 July.

Existing Conditions

50. Existing conditions can be determined from the field data collected. Figure 9 shows the height and orientation of the sand waves at locations beginning from the bay and progressing through the inlet into the gulf. The smaller sand waves in the bay are asymmetric, facing in a direction associated with flood-dominant currents. During the ebb-tidal phase, the flow

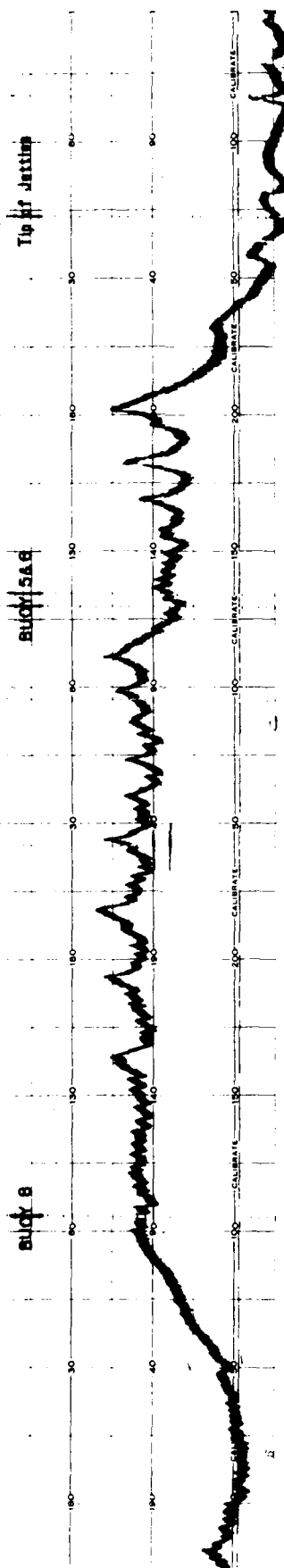
converges toward the inlet from all over the bay, and the resulting magnitude of tidal currents in this area is smaller than that on the flood tide. During the flood-tidal phase, currents are stronger because the inlet acts like a jet. As the flows exit the inlet throat, they begin to diverge and velocities decrease. However, according to data obtained near buoys 11, 12, 13, and 14 (Figure 2) the velocities are stronger than the converging ebb-tidal currents, which explains the flood-dominated orientation of these smaller sand waves.

51. Closer to the bay side of the inlet throat the sand waves are more symmetric. This symmetry is most likely due to more equal current conditions between ebb and flood. The diverging flood currents and converging ebb currents, approximately equal in magnitude, duration, or a combination of both, produce more symmetric sand waves.

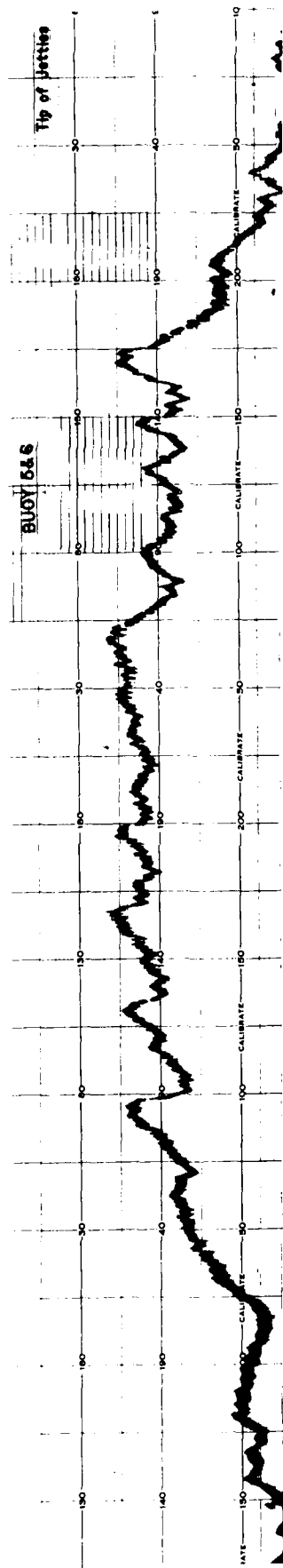
52. Within the inlet throat sand waves appear to be mostly ebb-tidal-dominated. Sand wave heights in this area average about 6 to 8 ft with extreme heights of 12 to 15 ft. These larger sand waves are found closer to the gulf end of the inlet and are more symmetric than the ones found toward the bay side. This difference in symmetry may be caused by effects of incident gulf storm waves or more equal ebb and flood tide conditions similar to those found near the bay side of the inlet throat.

53. Side-scan sonar surveys were used to provide information on the horizontal characteristics of the sand waves. As discussed in the previous section, the view is somewhat skewed because the sonar beam scans in off nadir directions. However, the image presented provides a qualitative picture of horizontal sand wave distribution. Figure 10 shows that sand wave crests extend across the inlet, perpendicular to the navigation channel. They appear to maintain a near constant elevation in the along-crest direction.

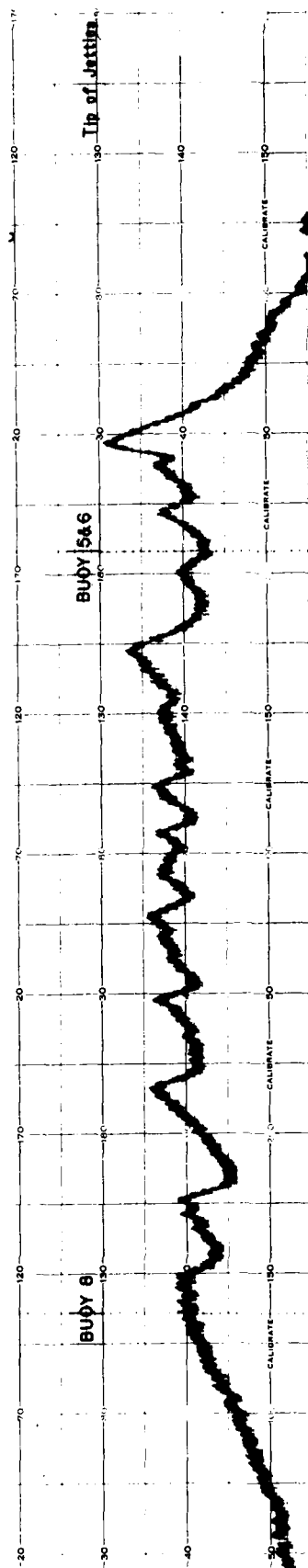
54. Horizontal spacing on various fathometer records is not constant between records or even from the beginning of one record to its end due to the dependency upon boat speed. Even so, comparing the fathometer recordings through time can provide a qualitative picture of sand wave growth. Figure 11 shows the navigation channel center line records from (a) 16 October 1986, (b) 6 November 1986, and (c) 10 July 1987. Only 5 months after the channel was dredged in May/June, sand waves were already evident (Figure 11) but were relatively small in both height and length. Approximately 3 weeks later the sand waves were somewhat larger in height and length and in number along the channel (Figure 11). Larger bed forms were visible, especially on the bay end



a. Record from 16 October 1986



b. Record from 6 November 1986



c. Record from 10 July 1987

Figure 11. Sand wave fathometer surveys

of the cut through Lands End Peninsula. A survey conducted on 10 July 1987 (one year after dredging) (Figure 11), showed that the sand waves had become still larger in height and length and more distinct in shape and orientation. The sand waves near the bay appeared strongly dominated by ebb currents, and those near the gulf entrance appeared more symmetric (equally affected by ebb and flood).

55. Tides at the study area are primarily diurnal, becoming more mixed during the 1/4 and 3/4 phases of the Moon. Maximum tidal range in the bay is about 1.5 ft, but storms can cause much larger water level fluctuations up to about 4 ft to occur. Ebb currents appear to be slightly larger in magnitude than flood currents. Maximum velocities measured during the ebb tide were about 2.8 ft/sec, and maximum velocities during the flood tide were about 2.3 ft/sec. During the period the gages were in place and recording data, the ebb-tidal phases were shorter than the flood phases by about 30 min. Weather conditions were mild with onshore winds of less than 10 knots.

56. Velocity distribution through the water column was found to be nearly constant except near the bottom. The boundary layer appeared to completely diminish approximately 5 ft off the bottom where velocities increased to those measured throughout the water column, providing excellent boundary conditions for the computer model described in the next section.

PART V: METHODS FOR MITIGATION

57. To most economically maintain the Panama City Entrance Channel the interval between maintenance dredging events must be maximized. To accomplish this, the formation rate and size of the sand waves must be reduced by (a) modifying inlet hydraulics to create flow conditions less favorable to sand wave growth, (b) reducing the amount of feeder sediment entering the navigation channel, and/or (c) developing an alternative dredging technique.

Hydrodynamic Analysis

58. A set of jetty, revetment, and channel modifications were evaluated using INLET, a spatially-integrated numerical model of inlet hydraulics (Seelig, Harris, and Herchenroder 1977). The model allowed simulation of the tidal currents through the entrance channel for each alternative tested. A generalized relationship between mean sediment size, mean flow velocity, and various bed forms (Figure 3) (SEPM 1975) was used to predict the bed forms likely to form for each modification. Each modification was evaluated in terms of whether it could reduce the formation of sand waves based on inlet hydraulics as predicted by Figure 3.

59. The numerical model INLET solves a spatially integrated one-dimensional momentum equation simultaneously with the continuity equation to predict water levels and currents for an inlet configuration. Grid cells, which are computational nodes in the model, are input to the model such that discharge is distributed between each cell to minimize friction. Input to the inlet model is a sequence of water level data at specified time intervals for the gulf and bay, area and length of each grid cell, area of the bay, forcing period and amplitude for a sinusoidal tide, two constants used in Manning's equation to predict friction, and the side slope of the inlet (see Appendix A for an example input file). Output from the model includes a sequence of velocity data for each cell in the inlet grid.

60. Hydraulics of the St. Andrew Bay entrance channel were simulated with four channels (parallel to flow) and five cross sections (perpendicular to flow), creating 16 grid cells. The dimensions for the grid cells were calculated using computer program NET (Seelig, Harris, and Herchenroder 1977) with input variables being cross-sectional dimensions and inlet discharge.

The program NET uses a weighting function to minimize friction, calculating location of grid boundaries so that flow through all channels will be equal. (See Appendix B for an example NET run.) Because of this function, grid cell dimensions varied with each inlet modification. The channels in the center of the inlet were closely spaced to account for higher frictional effects due to greater flow speeds. The model was calibrated using velocities measured during several spring tidal cycles on 10-13 July 1987 at navigation buoy 5 (Figure 2). Data were collected during these tidal cycles in order that maximum flood and ebb velocities could be measured. The model was calibrated by varying the two constants in Manning's equation for open channel flow until measured velocities matched predicted velocities. Predicted velocities from the calibrated model matched measured velocities well, as evidenced by a correlation coefficient of 0.98 (Figure 12).

MEASURED VS. PREDICTED VELOCITY

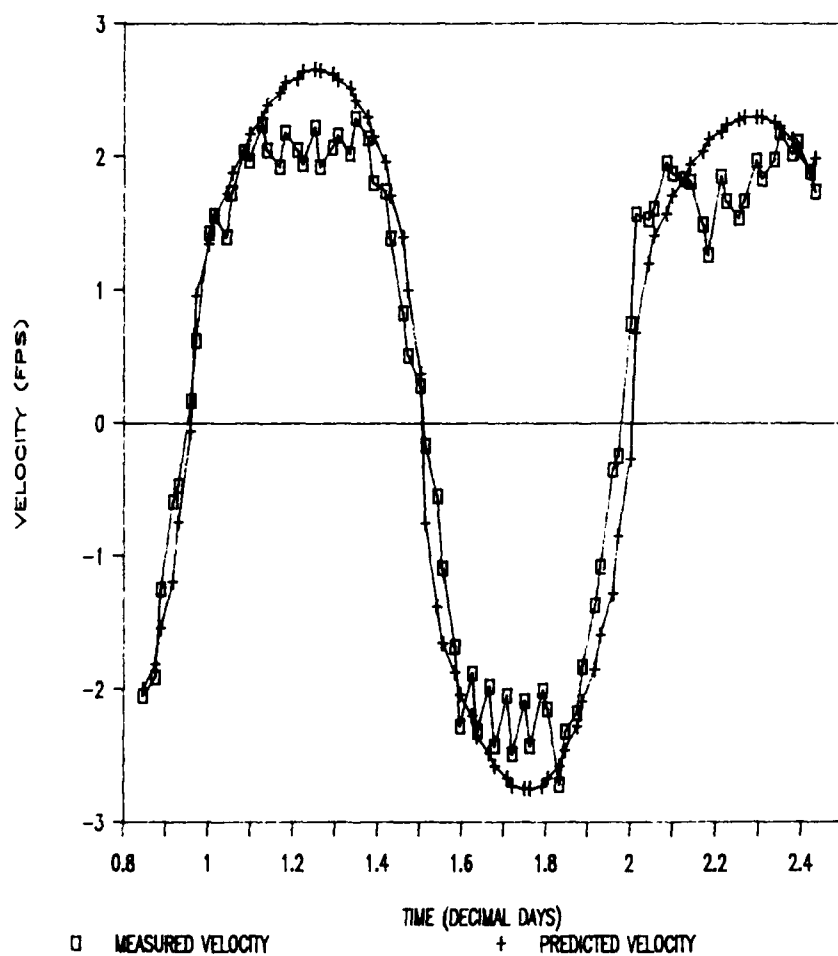


Figure 12. Velocity comparisons

61. Using the relationship between grain size and mean velocity to predict bed forms (Figure 3), the types of bed forms likely to form in the channel were predicted as a function of the velocities predicted by INLET for each configuration tested. The Panama City Entrance Channel is characterized by quartz sand with mean diameter varying between 0.2 mm to approximately 0.35 mm. Based on this grain size, reducing velocities in the channel to 20-55 cm/sec would modify the flow regime to a level associated with the formation of smaller bed forms such as ripples. Increasing channel velocities to 85-120 cm/sec would increase the movement of sediment in the channel, creating a flat bed geometry.

62. Sixteen modifications, involving various combinations of channel depth and jetty and revetment length and location were tested with INLET, and results are summarized in Table 5. Each modification modeled and the maximum and minimum velocities predicted at navigation buoy 5, the prototype control location, are presented in Table 5 and Figure 13; cases A through M represent modifications designed to reduce velocities to the ripple flow regime, and cases N through P represent modifications designed to increase velocities to the flat bed geometry.

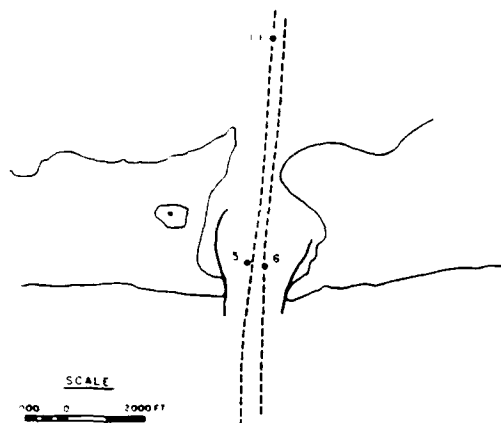
63. In an effort to reduce channel velocities, the original jetty configuration was modeled with channel depths increased to 40 ft and 55 ft (cases A and B). Predicted velocities included a maximum flood equal to 71 cm/sec (40-ft depth) and 68 cm/sec (55-ft depth); maximum ebb velocities were 76 cm/sec (40-ft depth) and 75 cm/sec (55-ft depth). The predicted velocities for both modifications are still above the ripple flow regime and thus are too great. Three modifications were evaluated with the channel widened 580 ft, 770 ft, and 980 ft (the maximum width allowed by the present jetty configuration). Each of these modifications was tested with channel depths of 40 ft and 55 ft; the 770-ft modification was also tested with a channel depth of 70 ft (cases C through I). Predicted velocities for these modifications again did not decrease velocities sufficiently for the lower flow regime. Lowest velocities occurred for case I, with a 1,375-ft width channel from offshore to the inlet entrance and 980-ft width channel from the inlet entrance to the bay. Predicted velocities were a maximum flood equal to 66 cm/sec (40-ft depth) and 55 cm/sec (55-ft depth); maximum ebb velocities were 70 cm/sec (40-ft depth) and 62 cm/sec (55-ft depth).

64. Further increases in channel width would require moving one or

Table 5
Inlet Modifications Tested Using INLET

Case	Description of Channel Configuration	Maximum Velocities cm/sec		Stable
		V(flood)	V(ebb)	
A	Original configuration, but channel at 40-ft depth	71	76	yes
B	Original configuration, but channel at 55-ft depth	68	75	yes
C	Channel widened to 770 ft from ocean to inlet, 580 ft from inlet to bay, 40-ft depth; original jetty configuration	70	75	yes
D	Same as Case C, but 55-ft depth	66	73	yes
E	Channel widened to 770 ft entire length 40-ft depth; original jetty configuration	70	74	yes
F	Same as Case E, but 55-ft depth	63	74	yes
G	Same as Case E, but 70-ft depth	54	62	yes
H	Channel widened to 1,375 ft from ocean inlet, 980 ft wide from inlet to bay, 40-ft depth; original jetty configuration	66	70	yes
I	Same as Case H, but 55-ft depth	55	62	yes
J	Channel widened to 1,275 ft entire length, 40 ft depth; west jetty spaced 1,700 ft	67	73	yes
K	Same as Case J, but 55-ft depth	55	64	yes
L	Channel widened to 3,125 ft entire length, 40-ft depth; jetties 830 ft apart	37	43	yes
M	Same as Case L, but 55-ft depth	27	30	yes
N	Original channel, 40-ft depth; inlet jettied through its length at approximately 1,400-ft spacing	78	82	yes
O	Same as Case N, but extend jetties offshore to 1,710-ft length	76	79	yes
P	Original channel, 40-ft depth; inlet jettied through length at approximately 1,100-ft spacing	88	91	yes

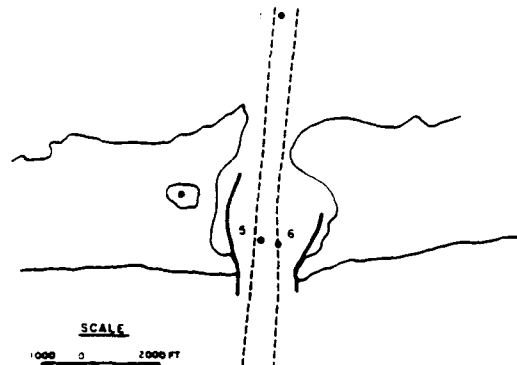
**ORIGINAL CHANNEL CONFIGURATION
40' & 55' DEPTH**



A & B

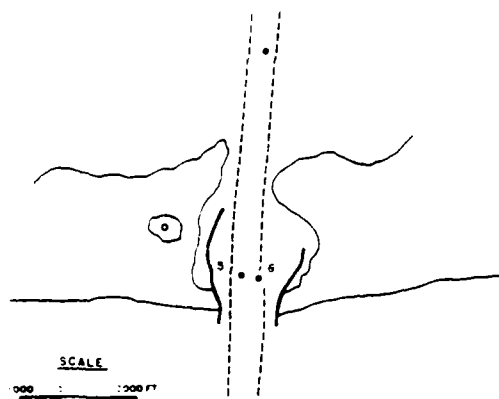
**CHANNEL WIDENED TO 770' ENTIRE LENGTH, 40',
55' AND 70' DEPTHS.**

**ORIGINAL CONFIGURATION, CHANNEL WIDENED TO
770' OCEAN TO INLET, 580' WIDE INLET TO BAY.**



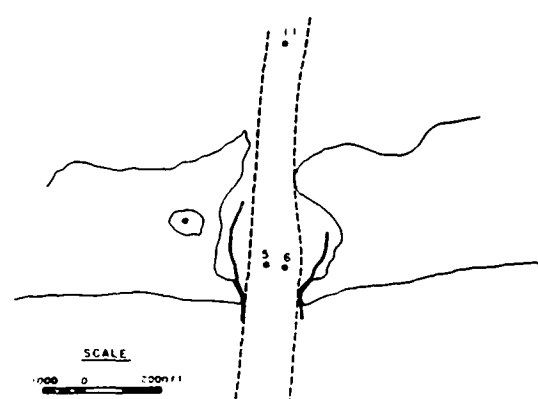
C & D

**CHANNEL 1375' OFFSHORE TO INLET, 980' WIDE
INLET TO BAY, 40' AND 55' DEPTHS.**



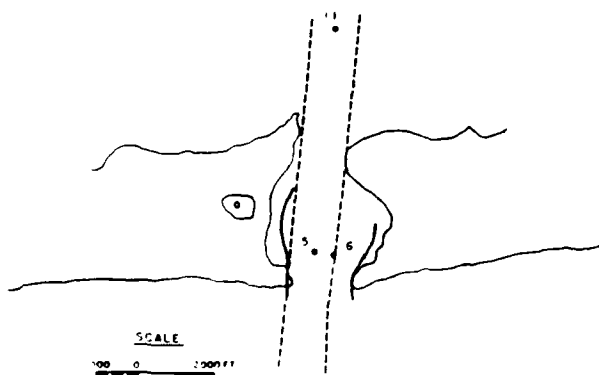
E, F, & G

**CHANNEL WIDENED TO 1250' ENTIRE LENGTH, WEST
JETTY MOVED WEST 190', 40' AND 55' DEPTHS.**

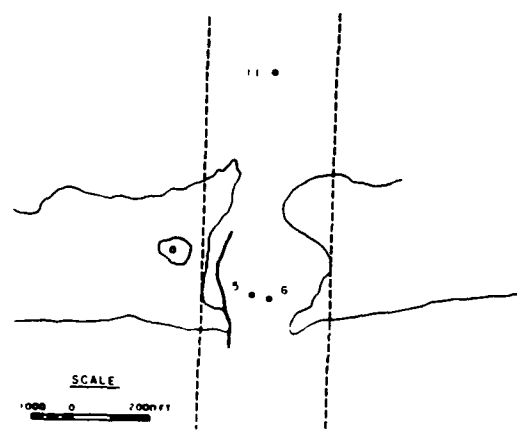


H & I

CHANNEL 3125' WIDE, 40' AND 55' DEPTHS.



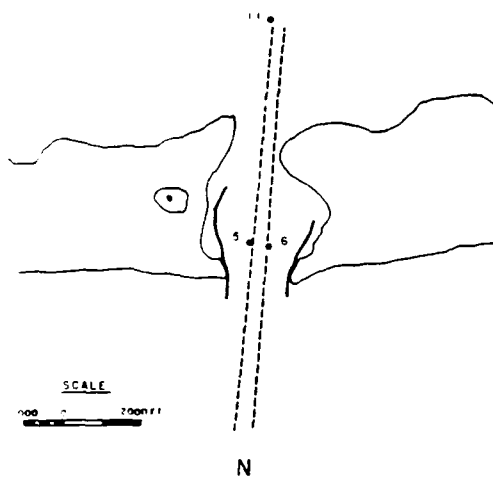
J & K



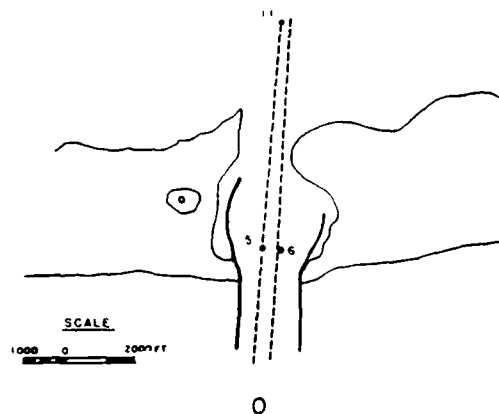
L & M

Figure 13. Inlet configurations (Continued)

REVETTED CHANNEL, 40' DEPTH



REVETTED CHANNEL, EXTENDED JETTIES, 40' DEPTH



REVETTED LENGTH OF CHANNEL, EXTENDED JETTIES,
DECREASE DISTANCE 250', DEPTH 40'

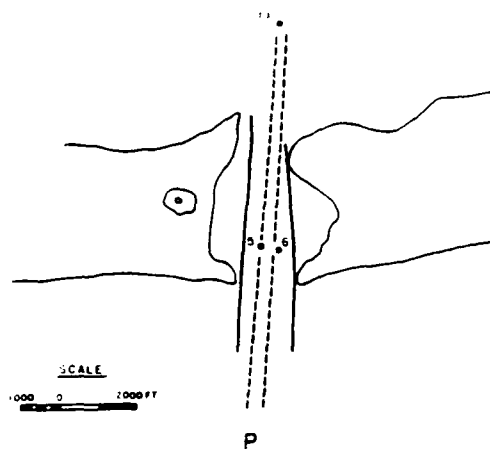


Figure 13. (Concluded)

both of the existing jetties. INLET was run for two such cases where the distance between the jetties was increased from 1,500 ft (present configuration) to 1,700 ft and 3,125 ft (cases J through M). The 1,700-ft configuration moved the west jetty approximately 200 ft to the west, with a 1,250-ft-wide navigation channel at 40- and 55-ft depths. Predicted velocities were a maximum flood equal to 67 cm/sec (40-ft depth) and 55 cm/sec (55-ft depth); maximum ebb velocities were 73 cm/sec (40-ft depth) and 64 cm/sec (55-ft depth). Both predicted velocities are above the ripple flow regime. The 3,125-ft configuration moved both jetties approximately 1,325 ft and allowed a 1,325-ft wide navigation channel at 40- and 55-ft depths. For this case, predicted velocities were in the ripple flow regime, with a maximum flood equal to 37 cm/sec (40-ft depth) and 27 cm/sec (55-ft depth); maximum ebb velocities were 43 cm/sec (40-ft depth) and 30 cm/sec (55-ft depth).

65. Decreasing the distance between the jetties (cases N through P) was also examined to increase velocities above the point at which an upper flat bed is obtained (Figure 3). For the first configuration, the entire channel was revetted along its length to streamline the flow, and a navigation channel depth of 40 ft was tested. The predicted velocities were a maximum flood equal to 79 cm/sec and a maximum ebb equal to 82 cm/sec which would not be sufficient to create the desired flat bed geometry. The jetties were then extended 1,710 ft, leaving the revetment along the length of the 40-ft depth channel as before. Predicted velocities were a maximum flood of 76 cm/sec and a maximum ebb of 79 cm/sec, again not in the flat bed flow regime. Reducing the jetty and revetment spacing to 1,250 ft, the flat bed flow regime was reached using the 40-ft depth navigation channel. Predicted velocities were a maximum flood of 88 cm/sec and a maximum ebb of 91 cm/sec.

66. Using predictions from INLET and Figure 3, cases L, M, and P could potentially reduce the formation of sand waves in the channel. The stability of each of these inlet configurations was evaluated using Equation 4-62b from the Shore Protection Manual (SPM) (1984). This equation relates the inlet tidal prism P to the minimum cross-sectional area required for stability A_c . This equation was empirically determined using Gulf of Mexico inlet data and is presented in the SPM as

$$A_c = 5.02 \times 10^{-4} P^{0.84}$$

67. The St. Andrew Bay tidal prism was calculated using the mean bay tidal range, area of the bay, and the ratio of Keulegan numbers at the Entrance Channel and Shell Crooked Island channel. Using the Keulegan ratio (0.83)* and bay prism ($1.93 \times 10^9 \text{ ft}^3$), the minimum cross-sectional area for stability is approximately $31,700 \text{ ft}^2$. Using this method, each of the three sand wave reducing inlet configurations tested would be stable.

68. Based upon the results of this study, it appears that significant modifications to the existing inlet and jetty configuration would be required to alter the hydraulic parameters to provide velocities less favorable to sand wave formation.

Sediment Supply Reduction

69. Three sediment sources were examined as major suppliers of sand which feed the sand waves. Windblown sand from adjacent shores and dune fields also contributes to the sand waves. This source is the smallest portion, as estimated using an equation developed by O'Brien and Rindlaub (Horikawa, Hotta, and Kraus 1986) for windblown sediment transport. Wind rose results for the study area (SAM 1976) reveal an estimated maximum contribution of 10,000 cu yd annually may be attributed to windblown transport.

70. A second potential source of sediment is from erosion of the channel banks within the inlet throat. Waves from the Gulf of Mexico diffract as they enter the entrance channel. These waves are probably the primary force causing erosion along the channel banks. The estimated annual contribution from this source is over 20,000 cu yd based on shoreline change rates measured from aerial photographs (1945 through 1981). A 1938 study** concluded that "the insufficiency of the length of the jetties and the divergence of the wings permit gulf storm waves to enter and cross the waterway and impinge upon the jetties and the unprotected shoreline, causing a progressive widening of the cut". Again, in 1946 a letter report from the War Department Shore Protection Board to SAM stated, "erosion of the banks of the New Channel presents a serious problem"† as related to channel shoaling.

71. This erosion problem still exists, especially along the shoreline

* R. G. Dean, 1971, from notes prepared for short course on inlets, Coastal Engineering Research Center, Ft. Belvoir, VA.

** See footnote on p. 17.

† See footnote on p. 18.

adjacent to the west jetty and extending north toward the bay. Recently the area was artificially nourished to prevent an upland freshwater lake from being breached. Inspection of this renourished channel bank showed a 3- to 4-ft scarp along much of the shoreline, indicating that erosion is continuing.

72. Longshore transport is the third and primary source providing sediment to the entrance channel sand waves. Several estimates of longshore transport rates were found in the literature and are summarized in Table 6. The short jetties and poor natural sediment bypassing characteristics of the inlet make it a natural sediment sink. From aerial reconnaissance and longshore transport estimates, most of the sediment entering the inlet appears to enter from the east. This occurrence agrees with the predominantly westerly direction of sediment transport for the region.

Table 6
Annual Longshore Transport Rates

<u>Source</u>	<u>Gross</u>		<u>Net</u>
	<u>East</u>	<u>West</u>	
Wang et al. 1978	300,000	510,000	210,000
Walton 1979	192,000	219,000	27,000
Stapor 1973	141,000	153,000	12,000
SAM 1976	413,000 (total gross)		6,800

Note: Measurements in cubic yards.

Dredging Alternatives

73. An alternative dredging technique does not necessarily require removal of sand from the entrance channel; rather, sediment could be redistributed throughout the area. The objective is to increase the maintenance dredging interval by conventional dredging, thus reducing costs associated with maintenance. Where sand waves exist, crest elevations determine the navigable depth in the channel. Removal of the crests (or tops) of sand waves would increase the interval between conventional dredging events. Removal of sand wave crests through agitation dredging techniques has been attempted by SAM and other Corps of Engineer districts with varying degrees of success.

74. Richardson (1984) reviewed over 13 projects involving agitation

dredging with a variety of equipment and conditions. The projects summarized dealt with localized shoaling, of which sand waves can be considered a subset. Several locations were maintained by deflection of vessel propwash which scours the bottom allowing currents to carry the material out of the area. This method was effective but only in shallow depths less than about 15 ft.

75. Dragging a rake or similar device was used at several locations with successful results. Rakes were used at Savannah Harbor to agitate silty bottom material in ship berths and terminals. Richardson states that at four sites where heavy shoaling occurred, 17 operations were carried out, each removing between 15,000 and 37,000 cu yd and increasing depths 2 to 3 ft. In China, several locations are maintained by dragging rakes modified with compressed air jets to increase the agitation process. However, the best results occurred at locations where the bottom material was silt or very fine sand less than 0.1 mm.

76. US Army Engineer District, Portland (NPP), maintains reaches along the Columbia River in Oregon which are prone to sand wave formation. Recently, NPP experimented with a new dredging technique to more effectively maintain their navigation channels. In the past, hopper dredges have been used to skim off the tops of sand waves. This method works; however, it is inefficient because over the troughs of a sand wave the dredge heads are moving through water and thus not removing bottom material. Also, it is not possible to change procedure by maneuvering the dredge along the sand wave crest. The river is narrow compared to the vessel length; and river currents make it difficult, if not impossible, to operate perpendicular to the direction of flow.

77. During the summer of 1987 NPP contracted with Rydel International to test a sand wave "skimmer" dredge. The idea behind the new technique was not to remove sediments from the problem areas. Instead, the skimmer was used to agitate sand into suspension so that river currents could carry it away from the crests, allowing suspended sediment to settle into the sand wave's trough. Figure 14 shows a generalized concept of the dredge. A large pump rests on a barge tended by a tug. A boom approximately 60 ft long with small jets is connected to the pump and adjusted by an A-frame device. The boom is pulled along the tops of sand waves in the direction of river flow.

78. River tests lasted 2 weeks, with periodic bathymetric surveys to monitor the progress of the new skimmer. The skimmer dredge was not able to move the pretest calculated volume. It appeared that sediment was not

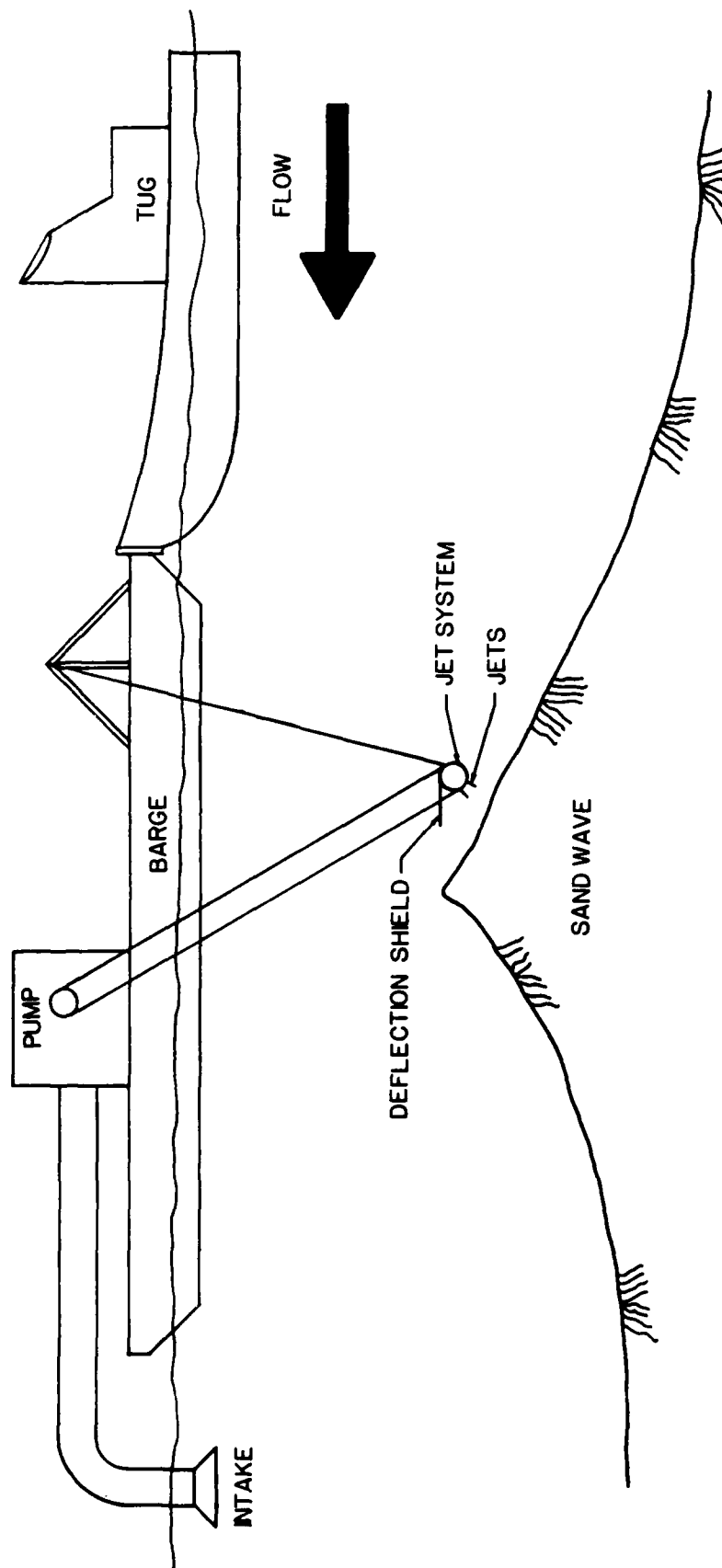


Figure 14. Experimental sand wave dredge

agitated high enough into the water column to allow currents to carry it far enough downstream to reach the wave troughs. The boom which moved across the sand wave crest slowly became impeded by sand buildup and acted more as a plow than an agitator. Partial cause is thought to be the very low river current velocities (near 100 year lows) which prevailed during the tests. NPP is planning to test the dredge again under higher flow conditions, perhaps in the summer of 1988.

79. Agitation dredging using rakes or similar devices appears to work best in locations characterized by silts and fine sands where the material can easily be suspended into the water column. However, larger sized sediments can be moved if enough force is applied, as in the case of deflecting prop-wash. At the Panama City Entrance Channel, where sediment sizes range from fine to medium sand, completely removing sediment from the channel is not necessary to increase the navigable depth. Moving the sand wave crests into the adjacent troughs by agitation dredging could reduce the maintenance dredging interval by conventional dredging. Therefore, a device such as the one being tested by NPP could prove effective in maintaining sand wave prone areas.

80. Specialized dredging techniques to maintain areas prone to sand waves are being studied under a new research work unit under the Improvement of Operations and Maintenance Techniques Research and Development Program. Results of this study should help in determining more cost-effective methods to maintain affected navigation channels.

PART VI: ENGINEERING OPTION EVALUATION

81. This report provides an initial investigation into sand waves and their related navigation and maintenance problems at St. Andrew Bay. It is beyond the scope of this study to provide a final design for mitigating these sand wave problems. Rather, the results provide engineering options available for further consideration and evaluation. Before any solution can be identified as final, a more complete evaluation must be undertaken which addresses both engineering and economic considerations.

82. The options presented are designed to increase the maintenance dredging interval by altering existing flow conditions through the inlet and/or by reducing sediment supply into the inlet. To study the inlet hydraulics and sedimentation, a hydraulic analysis and sediment budget was performed to determine if the navigation channel and inlet characteristics could be favorably altered, thereby producing more desirable conditions. This section discusses the various alternatives considered which were found to produce the desired bed conditions. The analysis did not include consideration of inlet navigability caused by each alternative. Further evaluation of each alternative is required before implementation.

83. By altering the physical parameters of the inlet, various alternatives were evaluated which either increased or decreased inlet current velocities. The alternatives modified existing conditions by altering either (1) channel dimensions or (2) jetty length and/or spacing.

84. In the previous section it was shown that modifying existing channel dimensions would not produce flow conditions conducive to smaller bed forms. Cases A through I (Table 5) indicate that widening and deepening the existing channel would not reduce velocities below approximately 62 cm/sec on ebb (55 cm/sec on flood). These velocities are above the lower limit for the smaller bed forms. In fact, modifying the channel to unrealistic dimensions such as case G (a 70-ft-deep channel) only reduces velocities to 62 cm/sec on ebb (54 cm/sec on flood).

85. Increasing the jetty spacing and channel dimension were considered and found not to produce favorable conditions. Cases J and K indicate that moving the jetties apart from 1,500 to 1,700 ft would not reduce velocities sufficiently to cause the smaller bed forms.

86. Cases L and M indicate that to significantly reduce inlet flow

velocities, unreasonable channel dimensions would be required. With the channel width changed to 3,125 ft (nearly three quarters of a mile) and depth of 40 ft, velocities reduce to 43 cm/sec on ebb (37 cm/sec on flood). With the 55-ft channel depth, velocities are further decreased.

87. Based on these results, it does not appear practical to alter the physical characteristics of the inlet in an effort to produce smaller bed forms. Cases N through P consider constricting inlet cross sections to cause greater flow velocities in the flat bed flow regime. Case N produces velocities just under the minimum velocity of 85 cm/sec required to reach favorable bed conditions. Case P, with the jetties spaced 1,100 ft apart, constricts flow to produce 91 cm/sec velocities on ebb (88 cm/sec on flood) necessary for a flat bed regime.

88. Moving the jetties 400 ft closer together would be a very involved construction effort. However this alternative, compared with widening the inlet nearly three quarters of a mile, is more feasible. A possible alternative to moving the jetties closer is construction of dikes along the east jetty and jetty wing. The dikes would constrict current flow and produce the desired velocities. In its present form, INLET cannot model such a system. Spacing and design of the dikes must be evaluated before this alternative can be fully considered.

89. Unless significant modifications are made to the existing inlet jetty configuration, altering the hydraulic parameters to provide velocities less favorable to sand wave formation is not practical. Reducing the sediment supply provides another method to limit formation rates and increase the maintenance dredging interval of these sand waves. Possible modifications to the jetties are presented below. They address the three major sediment sources and qualitatively describe methods to reduce the supply of sand reaching the sand waves. Additional study would be required to fully assess each alternative.

90. Raising the elevation of the jetties and existing revetments a few feet could reduce the windblown sand contribution from the estimated 10,000 cu yd annual quantity. However, the strategic placement of sand fences or vegetation plantings may also reduce this volume and be more economical.

91. Extending the existing east and west revetments toward the north and sand tightening them along both shorelines could greatly reduce the estimated 20,000 cu yd annually attributed to channel bank erosion. Special

care would be necessary around the "Kiddy Pool" area so as not to completely cut off the water supply and dry up this popular recreational attraction. The length and height of the revetments should be evaluated based on wave characteristics to determine specific dimensions and stone sizes.

92. Extending both jetties seaward would reduce longshore sediment transport into the inlet and reduce the channel bank erosion. However, extending the jetties may increase downdrift erosion if other provisions are not incorporated into the project. For example, artificially bypassing sediments around the existing jetties could reduce the sand supply to the channel and provide tangible benefits to the downdrift beaches in the form of beach erosion mitigation. If the decision is made to lengthen the jetties, the length would have to be determined based on wave and sediment transport characteristics and economics.

93. Because the natural channel thalweg and/or contribution from scour around the west jetty produce greater depths on the west side of the inlet, redefinition of the marked navigation channel to follow the natural channel could help to reduce maintenance requirements. Natural depths along the west jetty are actually deeper than those in the federally authorized navigation channel. Between the west edge of the channel and the west jetty is approximately 40 ft, which is sufficient room for a partial realignment. However, before such a change is made further evaluation of the channel is required. Also, possible increased scour along the west jetty and continued shoaling along the east side of the inlet must be considered.

94. This initial evaluation of the sand waves problem in the St. Andrew Bay entrance channel has identified several structural and nonstructural approaches. These alternatives are feasible in an engineering sense, but further investigations are required before final designs can be made and economic benefits calculated. From the results of NPP dredge tests, it appears that continued development is required before the sand wave skimmer will be efficient and ready for production level operation.

PART VII: CONCLUSIONS AND RECOMMENDATIONS

Possible Alternatives

95. From the preliminary analysis, there are only a few possible alternatives which could be implemented to reduce the sand wave dredging frequency. The next step to identifying an implementable solution is to further evaluate these alternatives and their effect on local coastal processes.

96. Constriction of the jetties, as discussed in paragraph 92, requires evaluation and design of dikes and their potential effect on the currents. The design requires structures large enough to withstand ambient conditions while not interfering with safe navigation. Construction materials and designs will need consideration to identify the most cost-effective combination. Also, the effect increased current speeds have on inlet navigability and possible scour around the jetties and proposed structures must be included in the analysis.

97. Extension of the revetment and/or jetties as a possible choice requires a thorough study of the related coastal processes. Reduction of the primary sediment source supplying sand to the sand waves requires extending the jetties. This extension is sure to cause adjacent shoreline response which could be numerically modeled, thereby prompting predictions of necessary levels of mitigation. Mitigation alternatives include constructing a sand bypassing plant at the inlet or selected placement of sand dredged from the inlet. A wave diffraction analysis is also required to estimate reduced channel bank erosion from each alternative.

Further Steps

98. To fully assess these potential mitigating alternatives through numerical or physical modeling, the acquisition of basic data is recommended to evaluate and better quantify the coastal processes and complex hydraulic and sediment interactions. For either potential alternative, constriction of the inlet or extension of the jetties, onsite wave and current data would assist in calculating wave diffraction through the inlet throat, computing rock and armor stone sizes, and quantifying local longshore sediment transport rates in the vicinity of the inlet. A directional wave gage, installed and

operated 1 year at a minimum, would help ascertain a seasonal cycle of conditions.

99. Inlet water currents play an important role in sediment transport and sand wave formation. Underwater current meters placed adjacent to the navigation channel, immediately after a dredging event and operated for 1 year concurrently with the directional wave gage, would provide hydraulic information on the forcing function of sand wave development. This basic information could be used to determine if sand wave formation is gradual or episodic in nature.

100. Tidal data across the inlet would be used to determine water surface elevation differences between the gulf and bay. These data correlate with current velocities through the inlet. Data from the directional wave gage can be used to ascertain water elevations outside the inlet. However, a tide gage is recommended inside the bay, near the inlet.

101. The final synoptic data set recommended would be bathymetric surveys of the inlet on a monthly interval. These surveys would provide information on inlet shoaling and sand wave development such as height, length, formation rate, and mechanism.

102. Before the costs and benefits of each alternative can be fully assessed, a more thorough study is recommended to provide greater understanding of the sand waves and interrelated coastal processes. This study would also help ensure that an implemented alternative does not cause greater problems than already exist.

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APPENDIX A
EXAMPLE INPUT FILE FOR NUMERICAL MODEL INLET

PANAMA	CITY	TEST?	DATA			
48.00 model run time (hrs)	60.00 time step (sec)	70.00 capture interval (min)	0 IDEBUG	number of points	time step (min)	
Body of Water No. → 1	10.93 Tidal Spring Period (hours)	1.30 amplitude of tide	70	30.00		
-0.50	-0.49	-0.47	-0.47	-0.37	-0.30	
-0.22	-0.12	0.02	0.10	0.22	0.35	
0.49	0.65	0.77	0.91	1.05	1.19	
1.31	1.44	1.55	1.67	1.74	1.82	
1.89	1.94	1.98	1.99	1.99	1.98	
1.94	1.88	1.79	1.69	1.57	1.43	
1.28	1.13	0.97	0.80	0.64	0.48	
0.32	0.18	0.06	-0.05	-0.15	-0.22	
-0.27	-0.30	-0.30	-0.29	-0.26	-0.23	
-0.18	-0.11	-0.04	0.04	0.13	0.23	
0.34	0.45	0.56	0.68	0.80	0.92	
1.03	1.14	1.25	1.35	1.44	1.53	
1.60	1.67	1.72	1.76	1.78	1.80	
1	area of bay (ft ²)	Beta (Z)	number of points	time step (min)		
Body of Water No. → 2	0.18E+10	0.00000	70	30.00		
0.10	0.11	0.13	0.17	0.22	0.29	
0.37	0.47	0.57	0.69	0.81	0.94	
1.08	1.22	1.36	1.50	1.64	1.78	
1.91	2.03	2.15	2.25	2.35	2.43	
2.50	2.55	2.59	2.61	2.62	2.61	
2.57	2.51	2.43	2.33	2.21	2.08	
1.93	1.77	1.61	1.45	1.28	1.12	
0.96	0.82	0.69	0.58	0.48	0.41	
0.35	0.33	0.32	0.33	0.35	0.39	
0.44	0.50	0.57	0.65	0.74	0.84	
0.94	1.05	1.16	1.28	1.40	1.51	
1.63	1.74	1.84	1.94	2.04	2.12	
2.20	2.26	2.31	2.35	2.38	2.40	
1245	Number of Channels	Number of Sections	Manning's Constants (calibrated)			
1.00000	0.080	0.0007	side slope of inlet			
1.6619.	4117.	4117.	16538.	cross-sectional area for each channel and section, computed by <u>NET.FOR</u> (ft ²)		
215191.	4353.	4353.	18014.			
312904.	3339.	3343.	14629.			
410986.	4910.	5207.	21639.			
541378.	79.	7752.	20903.			
1529.4	98.0	98.0	522.5	average width for each channel and section, computed by <u>NET.FOR</u>		
2463.2	103.6	103.6	577.5			
3403.7	79.6	79.6	477.1			
4287.3	94.4	104.7	657.6			
52022.8	1.9	185.4	1533.9			
1750.0	1750.0	1750.0	1750.0	grid lengths for each channel and section (ft)		
21625.0	1625.0	1625.0	1625.0			
31917.0	1917.0	1917.0	1917.0			
41250.0	1250.0	1250.0	1250.0			
0.00	1.00	0.00	1.00	1.00	1.00	
1	2	2				

APPENDIX B
EXAMPLE RUN FOR COMPUTER PROGRAM NET

\$ R NET

ENTER NUMBER OF INLETS

1 ← Number of inlets, N

ENTER NUMBER OF CHANNELS, IC AND INLET DISCHARGE, Q

4, 105000.0

IC=

4 Q= 105000. ← inlet discharge, Q (ft³/s)
↑ number of channels, IC

WEIGHT OF EACH CHANNEL SHOULD BE = 0.25

ENTER ND AND DELX

13, 104.0

1

number of depths across channel to be entered, ND
distance between depths (feet), DELX

ND=

13

DELX=

104. (FT)

SECTION ST A

ENTER EQUALLY SPACED DEPTH READINGS ACROSS CHANNEL

0., 24., 30., 32., 40., 40., 40., 40., 32., 32., 32., 32., 0.0 ← equally-spaced depths across channel (ft)

EQUAL SPACED DEPTHS, FT

2.0 26.0 32.0 34.0 42.0 42.0 42.0 42.0 34.0 34.0 34.0 34.0 2.0

AREA= 0.4139E+05 (FT²) WIDTH= 0.1248E+04 (FT)

↑ calculated area of channel (ft²)

↑ calculated width of channel (ft)

CHANNEL DIMENSIONS FOR EQUAL DISCHARGE

J	A	B	D
channel number { 1	0.16619E+05	0.52941E+03	0.31391E+02
2	0.41167E+04	0.98017E+02	0.42000E+02
3	0.41167E+04	0.98017E+02	0.42000E+02
4	0.16538E+05	0.52254E+03	0.31650E+02

0.24513E+00 0.25154E+00 0.25154E+00 0.25179E+00

calculated weighting functions for each channel

FRACTION OF FLOW FOR 10 EQUAL SPACED CHANNELS

0.007	0.036	0.073	0.209	0.224	0.220	0.090	0
0.060	0.060	0.020					

* User-entered data are underlined.